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The influence of organic matter on shear strength of a cohesive soil

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THE INFLUENCE OF ORGANIC MATTER ON SHEAR STRENGTH OF A COHESIVE SOIL

BY

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THESIS

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ABSTRACT

The shearing properties of an organic A horizon of Bryce clay were studied and compared with the properties of the same soil treated with hydrogen peroxide. The treatment was to remove the organic matter and isolate it as a variable. Tests were performed on artificially sedimented samples of the untreated, 75 percent untreated, 50 percent untreated, 25 percent untreated, and the treated soil.

The effective stress failure envelopes were found to be higher for the untreated soil than that of the treated. The elastic deformation, however, was larger for the untreated samples. The pore water pressure development due to shearing process were found to be higher for the treated soil.

The difference in properties are thought to be caused by the presence or absence of the organic bonds.

ACKNOWLEDGMENT

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CHAPTER I

INTRODUCTION

Organic soils are soils with a sufficient content of organic matter to affect the engineering properties of the soil but do not have the spongy structure of highly organic soils such as peat or organic muck. Generally an organic soil has a lower unit weight, higher Liquid Limit, and greater compressibility than an inorganic soil. Organic odor or a dark earthy color are identifying factors for an organic soil.

It has been the general opinion that organic soils have a lower shear strength than inorganic soils, and usually they have been removed and replaced by inorganic soils before construction. Even though organic soils are encountered rather extensively in nature, little research has been done to explore the fundamental physical properties of these soils. Schmidt (1965) used hydrogen peroxide to oxidize and remove the organic matter and study the effect of carbon content as a variable affecting engineering properties of a soil. Schrotberger (1966) used the same removing method in an unpublished research project for the National Science Foundation to investigate the effect of carbon content on the effective shear strength of a cohesive soil consolidated isotropically. His findings pointed out clearly a higher effective shear strength for the organic soil than the same soil treated with hydrogen peroxide. There are many other research publications on organic soil but all refer to the highly organic soils, and mostly stress the consolidation characteristics rather than shear strength parameters.

The purpose of this research project is to investigate the effect of carbon content on the effective shear strength parameters of a cohesive soil consolidated isotropically and anisotropically.

CHAPTER II

PROCEDURES

A. Soil Description

The soil chosen for the investigation was Bryce clay loam to clay (Wascher, Smith and Odell, 1951). It is a dark grey soil that is found on level to gently sloping areas of Iroquois County, Illinois. The parent material to a depth of at least 18 inches is mostly water deposited lakebed sediment of the Wisconsin glacial period. It has a high clay content and relatively high organic carbon content. The sample used for this project was obtained from the NW 1/4 of SW 1/4 of Sec. 19, T. 24, R. 13 W of Iroquois County, Illinois. It was found that the soil from 0 to 2 inches in depth had an average carbon content of 5.0%. From 2 to 4 inches the carbon content dropped to 4.8%, from 4 to 6 inches to 4.6%. The entire testing program was conducted using soil from 2 to 6 inch depth. The average organic carbon content was found to be 4.7% (Green, W. J., 1969). Other important physical properties of Bryce clay used in this project are summarized in Table 1.

B. Soil Preparation

Preparation of the soil for this investigation is explained in detail by Green, W. J. (1969). The soil was initially air dried and pulverized by a Lancaster PC Mixer until nearly all passed a #40 sieve. The soil passing the #40 sieve was then split in two portions. One portion was used

TABLE 1

Physical Properties of Bryce Clay Loam

and H_2O_2 Treated Bryce Clay

	Particle Size Distribution (%)			Atterburg Limits			Organic Carbon (%)	Specific Gravity
	Sand >.05mm	Silt 50-2μ	Clay <2μ	LL	PL	PI		
Untreated	17	47	36	60.3	40.0	20.3	4.7	2.57
75% Untreated				57.0	35.5	21.5		2.59
50% Untreated				53.3	33.6	19.7		2.62
25% Untreated				50.3	29.6	20.7		2.64
Treated	14	38	48	45.1	25.0	20.1	1.1	2.66

without further treatment (and will be hereafter referred to as the untreated soil). The remaining soil was treated to remove most of the organic matter.

To remove the organic matter, hydrogen peroxide was used. This technique was developed by Baver (1930). Schmidt experimented with different concentrations of hydrogen peroxide and time of reaction (Schmidt, N. O., 1965). The procedure finally adopted for this project is the one used by Green, W. J. (1969). The soil was treated in 100 gram portions; 100 ml. of 30% hydrogen peroxide solution was added to the soil and the mixture placed in a flask at a 50° C water bath. As the reaction progressed, more soil and hydrogen peroxide were added to the flask. Finally the carbon content was reduced to about 1.1% from an initial value of 4.7%. A listing of other pertinent physical properties of the treated soil is given in Table 1.

C. Preparation of Sample by Sedimentation

1. Apparatus:

The sedimentation apparatus consisted of a plexiglass cylinder with an outside diameter of 2 inches. It was machined within a close tolerance to an inside diameter such that the area was 10 sq. cm. A plexiglass top and ram guide and a base containing a porous stone and a water outlet were connected to the cylinder. Connections were made by wing nuts and threaded brass rods. The ram was made of 1/2 in. diameter stainless steel. It fitted through the ram guide in the top of cylinder and served to transmit load from the loading plate to the piston and sample. The piston was equipped with a porous stone which was connected to four drains embedded in the piston. A schematic drawing of the apparatus is presented in Fig. 1.

2. Preparation of Apparatus:

Before the soil and water were mixed the apparatus was prepared as follows: The porous stone for the bottom drain was saturated with distilled water. A filter paper was cut to size, moistened, and placed on the stone. Water was poured on the filter paper which drained through the passage saturating the bottom drain. The bottom drain was then closed when only a thin film of water was maintained on top of the filter paper. This was done to prevent expansion of the air trapped in the drain when the sample was de-aired.

A thin coating of silicone oil was carefully applied to the inside of the sedimentation cylinder. The cylinder was then attached to the base and top by means of wing nuts and threaded rods.

The top porous stone,set into the piston,was also saturated and covered with filter paper. Finally the piston walls and the ram were lubricated with silicone oil. Caution was required when applying the lubricant to the piston to avoid oiling on the stone.

3. Sample Preparation:

A series of trial samples were prepared, using the untreated soil with varying water content and initial dry soil weight. It was found that 140 gr. of untreated soil

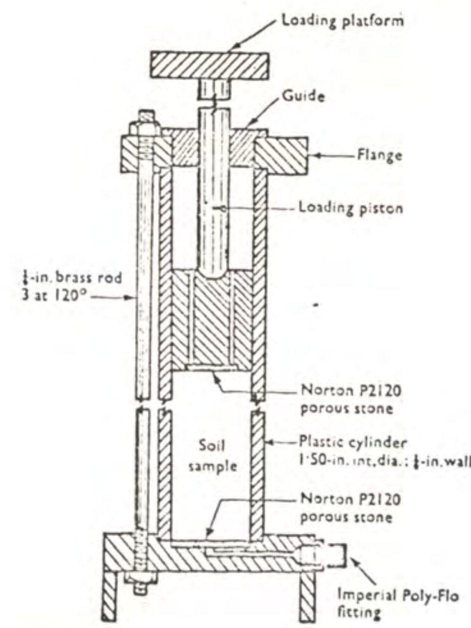


FIG. 1. CONSOLIDATION UNIT FOR SEDIMENTED TRIAXIAL SAMPLES

mixed with 160 ml. of distilled water made the best workable mixture. The mixture had to be thin enough so that it could be de-aired easily, but thick enough so that drainage would not require an excessive time and segregation of particles was avoided. It was found that the proper soil-to-water ratio was different for the treated soil; 140 gr. of soil to 150 ml. of distilled water. For different mixtures of untreated and treated soil the amount of water was interpolated between 150 and 160 ml.

Mixing of soil and water was done in a soil dispersion mixer. Distilled water was used at all times. The required mixture of untreated and treated soil was combined with the water in the soil dispersion mixer for 15 minutes. Using a funnel and plastic cylinder the slurry was carefully poured into the sedimentation cylinder. Drainage from the bottom was then blocked and a vacuum was applied to the top of the sedimentation cylinder to de-air the slurry. After air bubbles ceased to develop and no further removal of air was observed, the vacuum was removed. A rubber O-ring, slightly larger than the inside circumference of the cylinder, was then placed inside the cylinder to serve as a seal and to prevent the small soil particles from wedging between the piston and the cylinder wall. Next the piston was carefully inserted at the top of the cylinder. The rounded end of the stainless steel ram was then placed into the socket at the top of the piston and the latter guided to within a few centimeters from the top of the slurry. The platform was loaded very slowly to overcome the friction between the piston, O-ring, and the inside of the sedimentation cylinder. The piston would then move very slowly and make contact with the top of the slurry. The apparatus was allowed to set for about 5 minutes to allow initial sealing. Then an additional 6 Kg. weight was applied carefully to the loading platform. This load was constant during the entire testing period.

The sample was allowed to consolidate for seven days under an axial pressure of .6 Kg./sq. cm. At the end of consolidation the sample height was approximately 9 cm. After dismantling the apparatus, the sample was extruded by using the steel rod to push the piston. The sample was then trimmed by means of a wire saw to a length of 8.0 cm. in a cradle of the same length. The disturbance of the sample caused by extrusion could not be determined and was assumed to be negligible.

A hydrometer analysis was performed on the top and bottom half of a prepared sample to investigate the effect of segregation. The difference between the gradation of the top and the bottom portions of the sample was found to be negligible. Soil samples prepared in this manner were fairly stiff and easy to work with. Since the samples were to be consolidated further in the triaxial cell, any difference in water content in the sample's various sections would be eliminated.

D. Triaxial Compression Tests

The triaxial compression tests were performed utilizing a Geonor triaxial machine developed by the Norwegian Geotechnical Institute (Anderson, A. and Simons, N. E., 1960). A special rotating bushing in every cell was used to minimize the friction between piston and bushing. The complete description of the testing apparatus is given in the Geonor manual St. 22/63-AA/as.

All the compression tests performed for this investigation were Consolidated-Undrained with measurement of pore pressure. Samples were normally consolidated under either isotropic and anisotropic conditions. The test procedure is explained in two different steps; consolidation and shear.

1. Consolidation:

During consolidation in the triaxial cell, the sample was drained by means of a slotted filter paper placed around the sample and a porous filter stone on the bottom.

The friction between the ends of the specimen and the rigid end caps or porous stones restricts lateral deformation adjacent to these surfaces. Taylor (1941) indicated that end restraint does not affect the strength measurements provided that the length to diameter ratio of the sample is about 2. This was of some concern for this project because some of the samples when consolidated anisotropically had a length to diameter ratio as low as 1.6. To reduce the effect of end restraint and improve the method of testing no porous stone was used on top of the sample. Instead the top of the sample was lubricated with silicone oil and two separate thin rubber membranes were placed over the top end of the sample. Silicone was used to lubricate between the rubber layers and improve the lateral freedom of the movement of the top of the sample. The stainless steel loading cap was also lubricated and placed on the top of the sample. Bishop and Henkel stated that a ratio of length to diameter between 1.5 and 2.5 is satisfactory for strength measurements (Bishop, A. W., Henkel, D. J., 1962). Since none of the samples had a length to diameter ratio of smaller than 1.5, and the method described above was used to decrease the effect of end restraints, the length to diameter ratio of the samples was considered adequate. A single Trojan brand rubber membrane of .002 in. thickness encased each sample. Rubber Angus or O-rings sealed the sample at the top and bottom. The above procedures had to be done with extreme care to prevent sample disturbance. The pore water inflow or outflow was observed to be negligible. De-aired water was used in the cell so that air leakage through the membrane was eliminated.

The drainage tube leading from the porous stone at the bottom of the sample was connected to a 50 ml. burette filled with water. An initial burette reading was taken. Consolidation pressures were applied utilizing a dash pot to maintain constant hydrostatic pressure. A Geonor 22176 device was used to apply the extra vertical pressure necessary for anisotropic consolidation. This is shown in Fig. 2 .

FIG. 2. ANISOTROPIC CONSOLIDATION DEVICE

Loading for this isotropic consolidation was done in one step. However, for the case of anistropic consolidation, this was not practical. Loads were applied in small increments such that the ratio of vertical to horizontal consolidation pressures were always kept at 1.50. This was done during a period of 48 hours to prevent sample failure due to excessive vertical load. The final burette reading was taken after the completion of consolidation process. The difference between the initial and final burette readings indicated the volume change due to consolidation.

A separate triaxial consolidation test using porous stones at the top and the bottom of the sample was performed to determine the time necessary for full consolidation. A filter paper was placed around the sample but 1 cm. short of the bottom pore stone. The cell pressure was increased and the induced pore water pressure was applied through the top cap. Both pressures were equal to 2.0 Kg./sq. cm. After two hours the cell pressure was increased to 5.0 Kg./sq. cm. creating an effective consolidation pressure of 3. Kg./sq. cm. Drainage of the sample was through the top cap and into the system in which the dashpot induced the initial pore pressure. Dissipation of the pore pressure was measured at the bottom stone, using a C.E.C. Transducer type 4-312-0001. It was found that a period of 12 hours is necessary to achieve 98% pore pressure dissipation. However, all isotropic consolidation samples were allowed to consolidate for 24 hours. Those samples consolidating under anisotropic conditions were kept under the consolidation pressure for 24 hours after the last increment of load was applied.

2. Shear:

After completion of the consolidation process, the triaxial cell was placed on the Mossco's loading press. The drainage tubes to the burette were disconnected and de-aired water was flushed through the drainage tubes to remove any possible air trapped therein. One of the drainage tubes was plugged by a conical pin. To measure the increase in pore pressure, the other tube was connected to a pressure transducer cell of C.E.C. type 4-312-0001. This was done cautiously to prevent air bubbles in the tube or adjacent to the transducer drum. The transducer cell was then locked into a secure position on the triaxial cell. At that time all the drainage ways were closed and the sample was prevented from further consolidation. Back pressure was then applied by increasing the triaxial cell pressure. The system was left for approximately 5 hours to maintain equilibrium between the confining pressure and the pore pressure. The advantage of using back pressure is to dissolve air that may be trapped in the drainage tubes or between membrane and the sample (Bishop and Henkel, 1962). It was found by trial that a back pressure of 2.0 Kg./sq. cm. was adequate to secure 100% saturation.

In practice it has been found that a testing time of 4 to 6 hours to failure is sufficient to accurately measure the pore pressure of failure (Bishop and Henkel, 1962).

Since some samples were consolidated anisotropically and it was anticipated that the maximum deviator stress would develop at a very low strain (Anderson and Simons, 1960), the lowest rate of strain available on the loading press was used. All tests were performed at a constant strain rate of .134 cm./hr., which is approximately between 1.8% and 2.1% of the sample height before shear process per hour.

A proving ring equipped with an extensometer dial gauge for measuring the deformation of the ring was used to apply the axial force. The proving ring deflection was calibrated and the axial stress difference on the sample was simply calculated. The loading ring was placed into position under the loading yoke. The loading piston was then brought into contact with the proving ring, being held upwards by cell pressure. With the piston not in contact with the loading cap, the proving ring deformation dial indicator was then set equal to zero while the loading press was operating at its final strain rate. The piston was then carefully brought into contact with the loading cap. The strain gauge attached to the proving ring was then set to zero. By reading the strain indicator connected to the transducer the initial pore pressure was recorded. It should be noted that the above procedure had to be modified slightly for the tests performed on samples consolidated anisotropically. As back pressure was applied, a sufficient load had to be added to the anisotropic consolidation device to prevent the effect of upward pressure due to the cell pressure increase. While setting the proving ring on the sample all the dead loads on the anisotropic device were removed and a force equal to these loads was placed on the sample through the proving ring prior to setting it to zero. During the loading readings of the pore water pressure, proving ring deflection and axial deformation of the sample were taken at intervals until 20 to 25% strain was reached. Since drainage was not allowed during shear, the process proceeded with no volume change so that area correction was made according to the equation:

$$A = Ao/(1\text{-strain})$$

Ao is the initial horizontal area of the sample.

Two failure criteria were considered for analysis. They were defined by maximum deviator stress and maximum effective stress ratio. These two criteria of failure are the most commonly accepted definitions by engineers.

Upon completion of the testing, the sample was removed from the cell. Final height was measured and the sample was cut into three sections for which the water content was determined and the void ratios were calculated.

Since no drainage was allowed during the shear, the water content and the void ratio of the sample was considered to be the same as the initial condition before the shearing process started.

All the necessary calculations of the data were processed by an IBM 360/50 computer. A sample program and test data are presented in Appendix 2.

E. Testing Program

A minimum of 30 tests were scheduled for this project. Tests were performed on soil mixtures having five different carbon contents. Treated and untreated soil were mixed by the proportion of 0, 25, 50, 75, and 100 percent of untreated soil. A minimum of six tests were performed on the soil of each carbon content. Three of these were consolidated isotropically under 1, 2, and 3 Kg./sq. cm. all around pressure. The remaining three were consolidated under an anistropic condition with vertical to lateral pressure ratio of 1.5. The lateral consolidation pressure for these tests were also 1,2, and 3 Kg./sq. cm. The entire testing program was performed in a controlled atmosphere room at 20° C. temperature.

CHAPTER III

REVIEW OF LITERATURE

There has been very little research done on the subject of shear strength of organic soils and the effect of organic materials on the behavior of these soils under stress. Schrotberger (1966) investigated the effect of organic matter on the shear strength of a cohesive soil. He performed isotropically consolidated undrained triaxial tests on Paulding clay from Ohio. His findings are presented in an unpublished report to National Science Foundation. The following is a review of the results mentioned in that report.

A. Stress, Pore Pressure, and Strain Relationship

It was noted that for untreated soil, pore pressures did not exceed the deviator stresses. However, for the 50-50 mixture of hydrogen peroxide treated and untreated (organic) soil, the pore water pressure exceeded the applied stress difference for confining pressures of more than 5.0 Kg./sq.cm., i.e. Skempton's A coefficient of larger than 1 (Skempton, 1954). In treated soil this was true for almost all of the confining pressures except those smaller than .5 Kg./sq. cm. This phenomenon was interpreted to be caused by soil structure breakdown during shear. The treated soil was believed to have a weaker structure than the untreated soil. Hence, higher pressure had to be taken by the pore water.

For treated soil, the maximum effective stress ratio occurred at higher strains than did the maximum deviator stress.

B. Mohr Diagrams

It was concluded that the effective stress friction angle of treated soil was clearly smaller than that of untreated soil. Since shear strength is an increasing function with increasing effective angle of internal friction at the same effective stress level, the shear strength of the untreated soil (high organic content) appeared to be higher than that of the treated soil (low organic content). The effective stress friction angle of 50-50 mixture was very nearly that of the untreated soil. This condition was the same for both failure criteria; maximum deviator stress and maximum effective stress ratio.

C. Water Content and Void Ratio at Failure Versus Consolidation Pressure

During the same time period of consolidation process, the treated soil underwent a greater volume change and therefore had a lower void ratio and water content than untreated soil during shear. However, the untreated soil required a higher stress difference applied to failure. It was therefore concluded that the strength contribution of the organic material in the untreated soil was greater than the strength obtained by the denser treated soil.

D. Pore Water Pressures at Failure

Higher pore water pressures were developed in treated soil than in untreated soil. The pore water pressure at failure for 50-50 mixture was almost the same as for treated soil. The portion of the applied stress difference taken by the pore water was thus greater for treated soil than for untreated soil.

CHAPTER IV

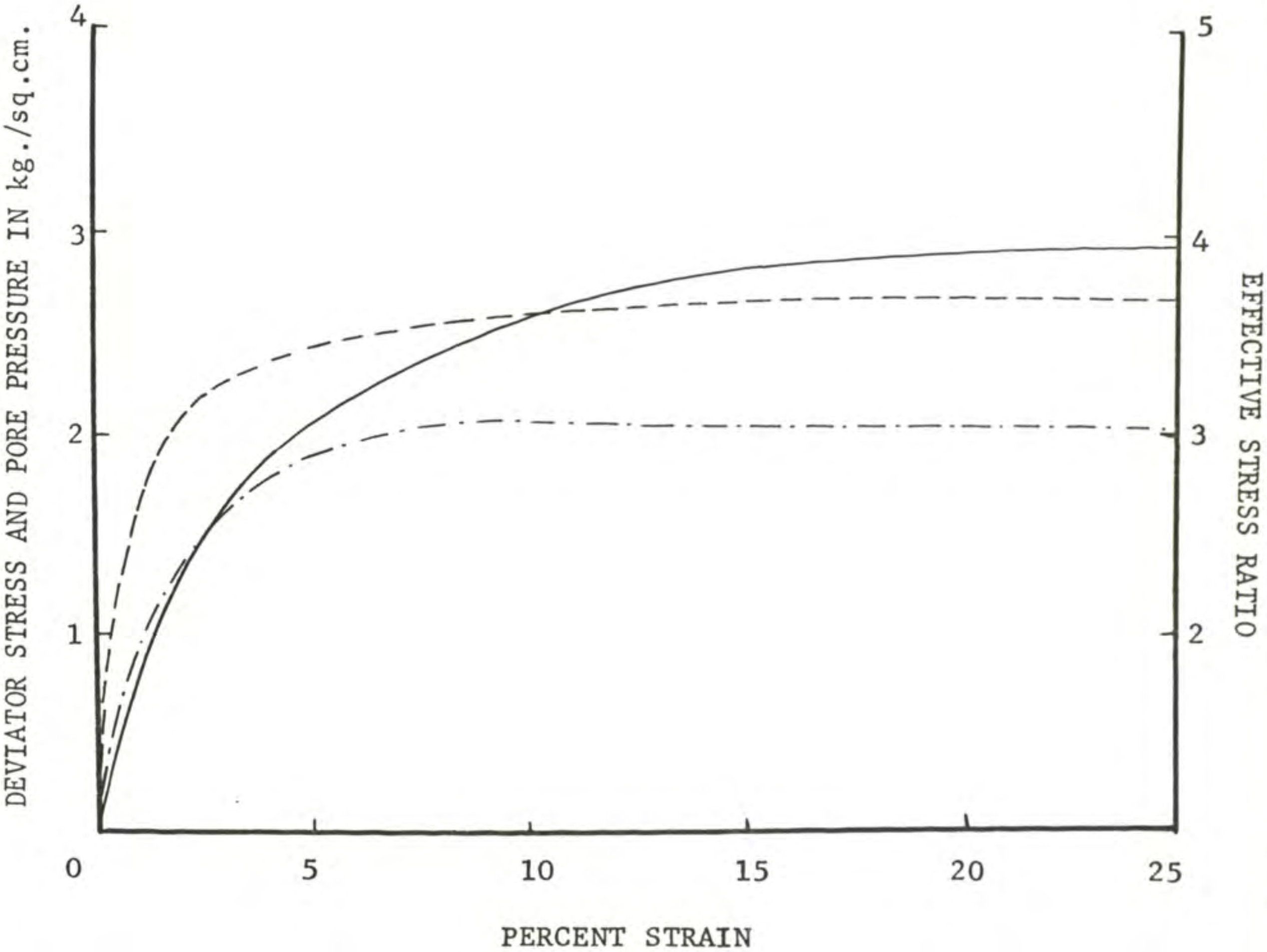
TEST RESULTS AND DISCUSSION

The results of triaxial shear tests on the treated (inorganic) and the untreated (organic) soils indicated different behavior for these soils at both states of isotropically and anisotropically consolidated samples. The results are presented under three general topics of stress-strain, strength, and the pore water pressure behavior.

A. Stress-Strain Behavior

In figures 3a through 3d samples of stress-strain behavior of the treated and untreated soils are presented for both isotropically and anisotropically consolidated samples. Effective stress ratio, axial stress difference, and pore pressures are plotted versus percent strain for these typical tests.

The straight line portion of the deviator stress versus strain plots indicates an approximately 40% greater deformation modulus at the same stress difference for the untreated soil than for the treated soil. Therefore, it is concluded that the immediate settlement of the untreated soil is greater than the immediate settlement for the treated soil. This might be a reason behind the general attitude that the organic soils are weaker soils than inorganic. This behavior is the same for both states of consolidation, i.e. isotropic and anistropic.



EFFECTIVE STRESS RATIO

DEVIATOR STRESS

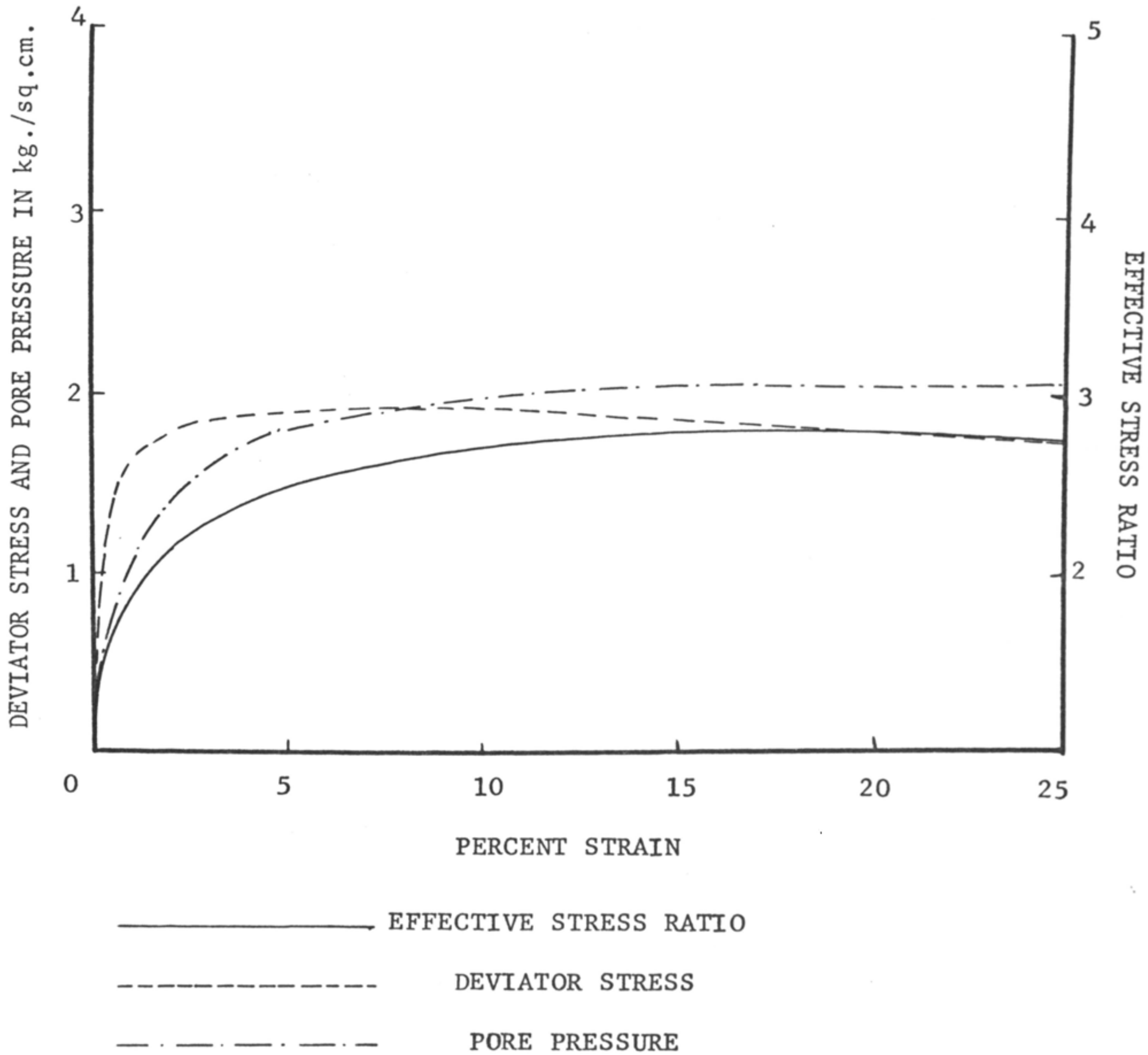
PORE PRESSURE

VERTICAL CONSOLIDATION PRESSURE = 3.0 kg./sq. cm.

LATERAL CONSOLIDATION PRESSURE = 3.0 kg./sq. cm.

PERCENT OF UNTREATED SOIL = 100.0

FIG. 3a DEVIATOR STRESS, PORE PRESSURE, AND STRESS RATIO VERSUS STRAIN.

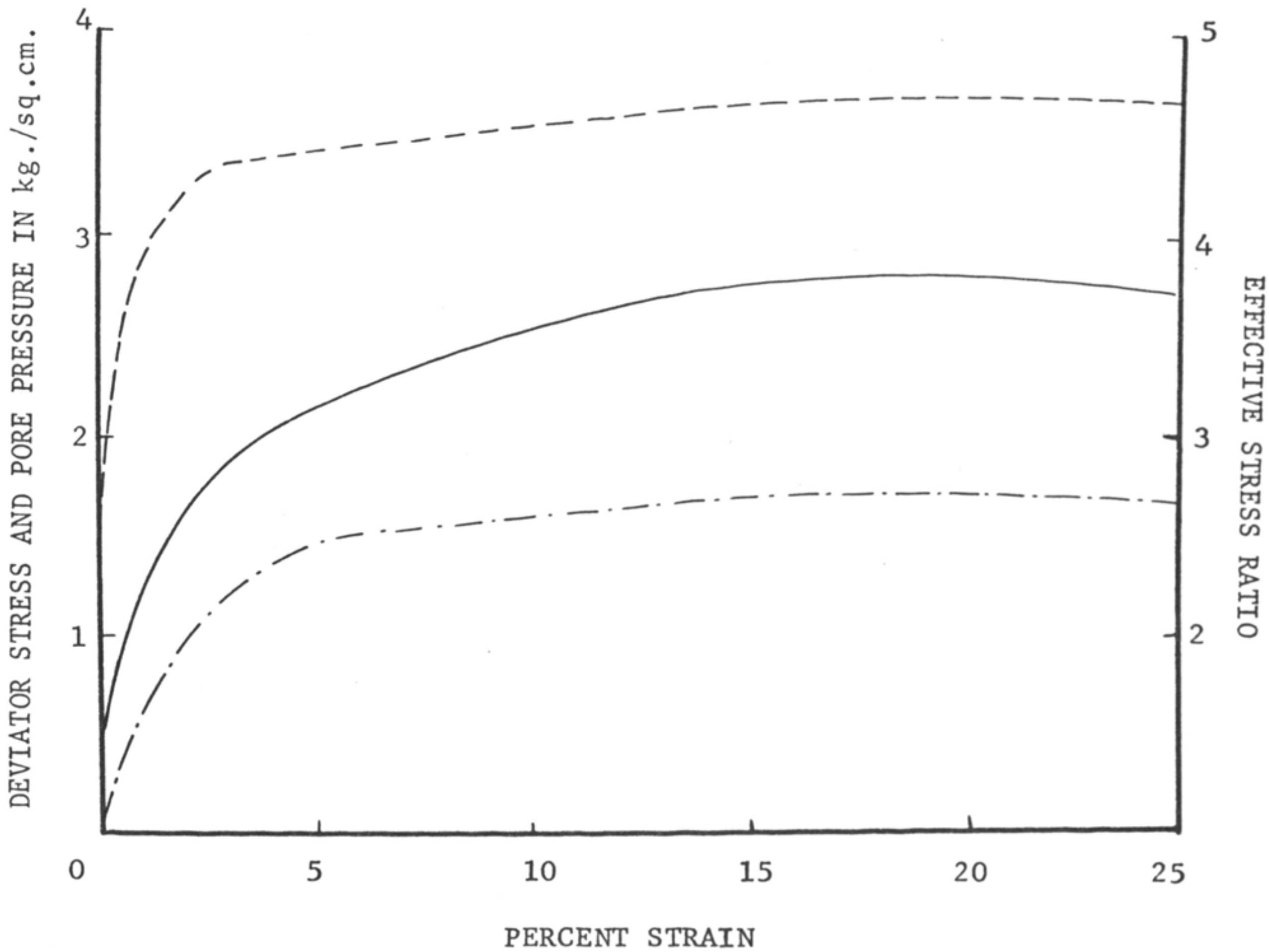


VERTICAL CONSOLIDATION PRESSURE = 3.0 kg./sq. cm.

LATERAL CONSOLIDATION PRESSURE = 3.0 kg./sq. cm.

PERCENT OF UNTREATED SOIL = 0.0

FIG.3b DEVIATOR STRESS, PORE PRESSURE, AND STRESS RATIO VERSUS STRAIN.



———————— EFFECTIVE STRESS RATIO

- - - - - - - - - - - - DEVIATOR STRESS

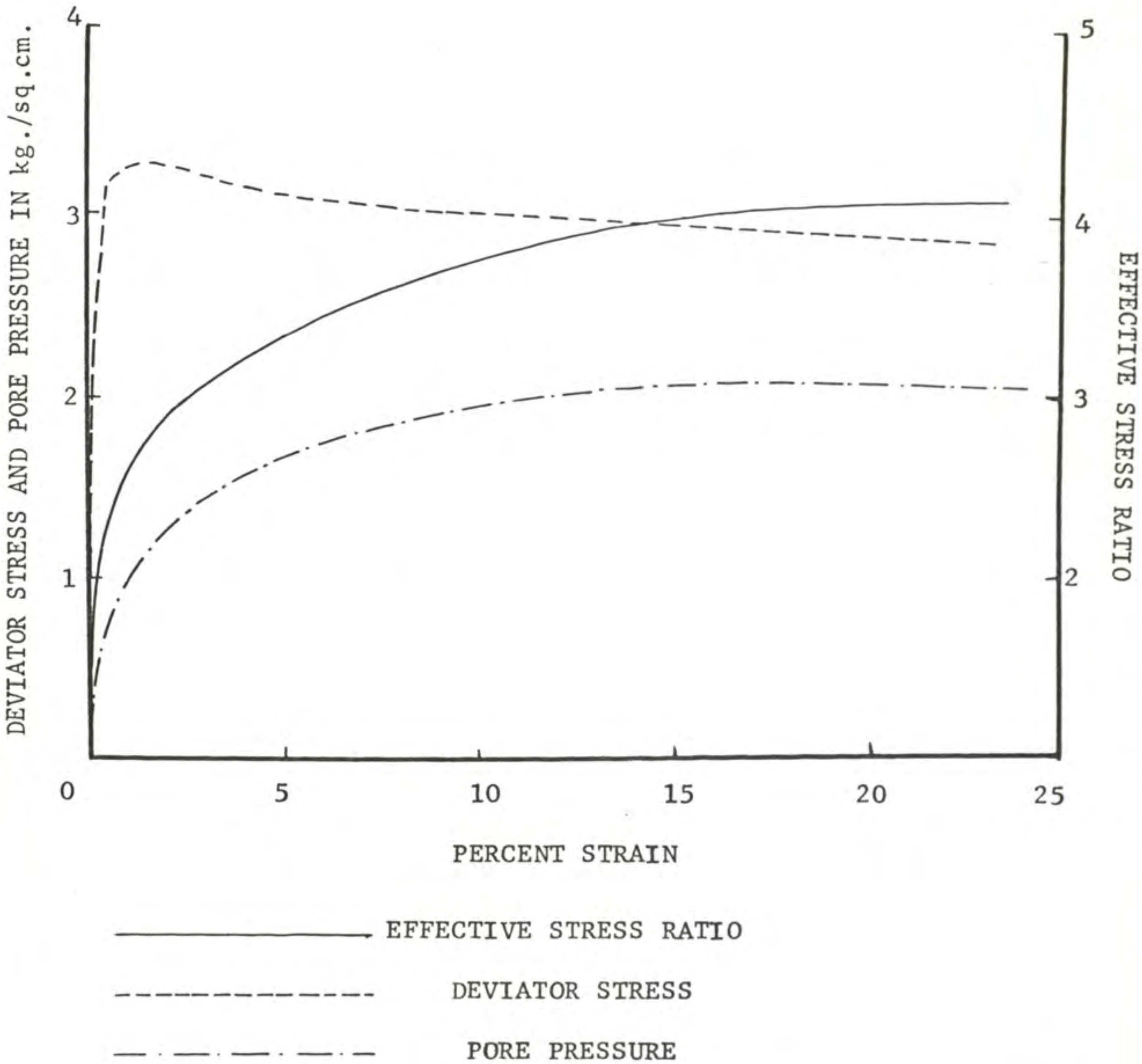
— · — · — · — PORE PRESSURE

VERTICAL CONSOLIDATION PRESSURE = 4.5 kg./sq. cm.

LATERAL CONSOLIDATION PRESSURE = 3.0 kg./sq. cm.

PERCENT OF UNTREATED SOIL = 100.0

FIG. 3c DEVIATOR STRESS, PORE PRESSURE, AND STRESS RATIO VERSUS STRAIN.



VERTICAL CONSOLIDATION PRESSURE = 4.5 kg./sq. cm.

LATERAL CONSOLIDATION PRESSURE = 3.0 kg./sq. cm.

PERCENT OF UNTREATED SOIL = 0.0

FIG. 3d DEVIATOR STRESS, PORE PRESSURE, AND STRESS RATIO VERSUS STRAIN.

B. Strength Behavior

1. Failure Criteria:

For the isotropically consolidated samples, the maximum effective stress ratio of the treated soil occurs at higher strains than does the maximum applied stress difference. The difference in behavior, however, is not as pronounced for the untreated soil samples and the other mixtures of the treated and untreated soils. This behavior can be observed in the data presented in Appendix 3. The same behavior occurs for the samples consolidated anisotropically, except that the maximum stress difference occurs at a much lower strain for the treated soil than for the same soil consolidated isotropically. Before the testing program started, it was anticipated that the samples consolidated anisotropically would reach the maximum stress difference applied at a smaller strain than that for maximum effective stress ratio. However, test results show that the untreated soil and 75 percent untreated soil samples do not behave as anticipated and the strains to failure for both conditions of consolidation are practically the same. But failure strains to the maximum stress difference for anisotropically consolidated soils were significantly smaller than the strains to the maximum stress ratio for the treated soil and 25 percent untreated soil. The 50 percent untreated samples also behave the same way, but the effect is not as pronounced as for the treated soil. The treated soil, in general, fails at lower strains than the untreated soil regardless of the condition of consolidation when failure criteria is the maximum stress difference. The failure strains of the treated and untreated soils, when the failure criteria is the maximum stress ratio, are practically the same.

2. Mohr Diagrams:

Instead of plotting the Mohr failure circles, it was found more convenient to plot $(\bar{\sigma}1-\bar{\sigma}3)/2$ against $(\bar{\sigma}1+\bar{\sigma}3)/2$ at failure, i.e. the top point of each failure circle. If α and ψ are the cohesion intercept and the slope angle for the straight line drawn through such points it can be shown that

$$\sin \Phi' = \tan \psi$$

and

$$C' - \alpha/\mathrm{Cos}\ \Phi'$$

The best straight line visually selected is passed through the peak points of the failure circles. When the scattered data points make the selection of the straight line difficult, the decision is influenced by the fact discussed hereinafter that the friction angle for the isotropically and anisotropically samples should be similar to each other.

In figures 4a through 4t, the summary of shear test data are presented for all tests using both maximum effective stress ratio and maximum deviator stress failure criteria. The effective angle of internal friction, Φ', values are summarized in Table 2. Small cohesion intercepts are found for all tests. This is thought to be caused by errors introduced by the necessity of measuring the pore pressures at the bottom of the sample rather than at the center. The cohesion intercept is larger for the treated soil and especially when consolidated anisotropically; in which case the failure, by both criteria, occurs at smaller strains and allows less time for the pore pressure to develop.

It is observed that there is little difference in the Φ' values for each mixture regardless of whether the sample is isotropically consolidated or anisotropically consolidated. This is to be expected because if Φ' is the angle of internal friction between the particles, its values should not depend on the state of consolidation. It is also observed that the value of Φ' for the untreated soil, 75 percent untreated, and 50 percent untreated are practically the same. However, the Φ' values are somewhat less for the 25 percent untreated and treated soils. It shows that small amount of carbon content has a surprisingly strong affect on the shear strength of the soil based on the magnitude of angle of internal friction between the particles. This effect, however, is not as clear after the carbon content increases over certain amount. It should be mentioned that the difference in Φ' values for the treated and the 25 percent untreated samples were not as large as the difference between the 25 percent and 50 percent untreated soils. This might be due to some experimental errors, or the possibility that the effect of carbon content

TABLE 2

VALUES OF Φ' AND C'

	Consolidated Isotropically				Consolidated Anisotropically			
	$(\bar{\sigma}1/\bar{\sigma}3)$ max		$(\bar{\sigma}1-\bar{\sigma}3)$ max		$(\bar{\sigma}1/\bar{\sigma}3)$ max		$(\bar{\sigma}1-\bar{\sigma}3)$ max	
	Φ' Degrees	C' kg/sq cm	Φ' Degrees	C' kg/sq cm	Φ' Degrees	C' kg/sq cm	Φ' Degrees	C' kg/sq cm
Untreated Soil	32.8	0.13	32.2	0.15	32.2	0.06	32.2	0.06
75% Untreated	32.2	0.12	32.3	0.12	32.0	0.06	32.2	0.06
50% Untreated	32.2	0.12	32.2	0.12	32.2	0.02	32.2	0.04
25% Untreated	25.2	0.18	25.2	0.16	25.8	0.21	25.2	0.11
Treated Soil	23.8	0.12	21.4	0.12	23.8	0.19	21.4	0.20

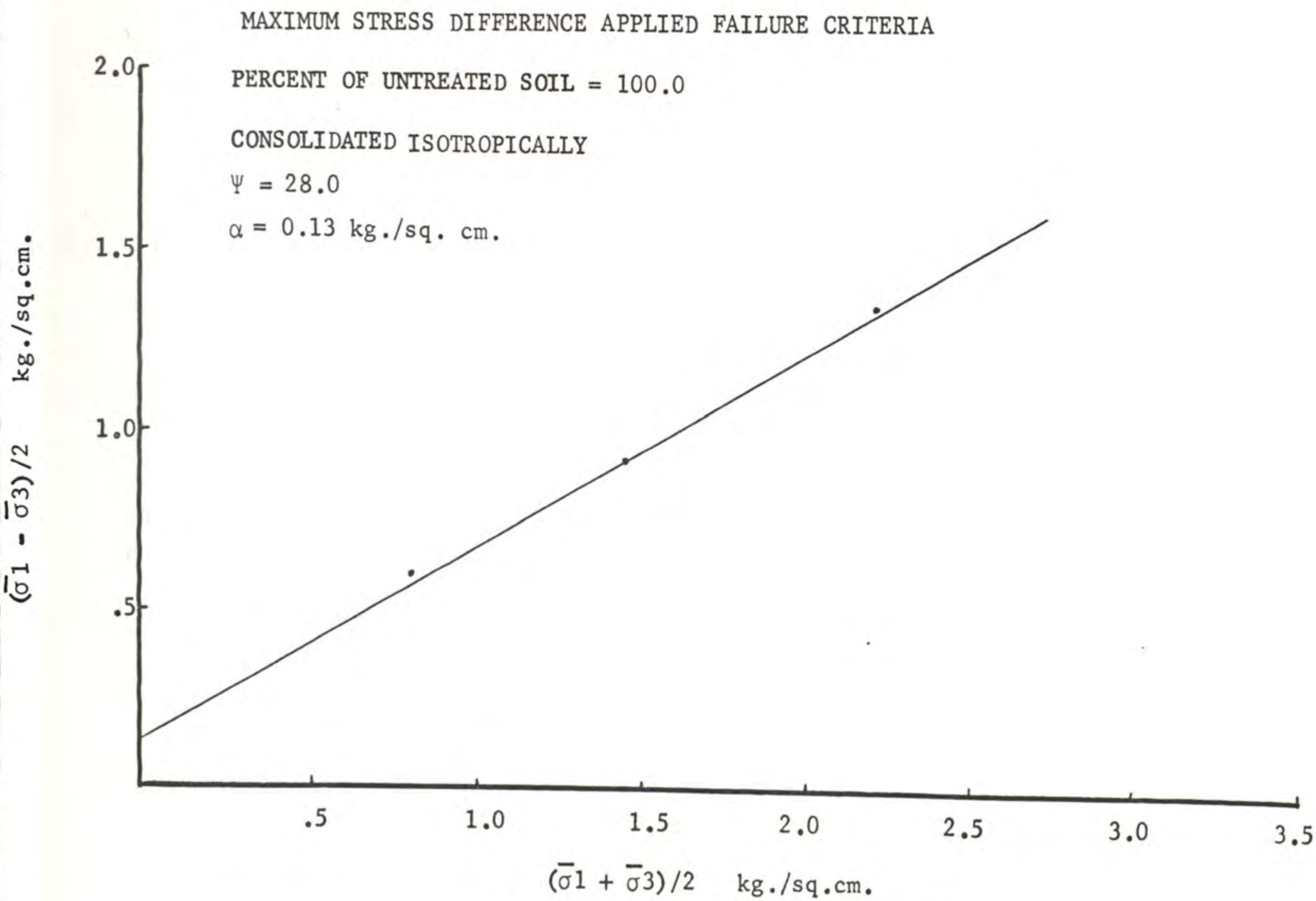


FIG.4a SUMMARY OF SHEAR TEST DATA

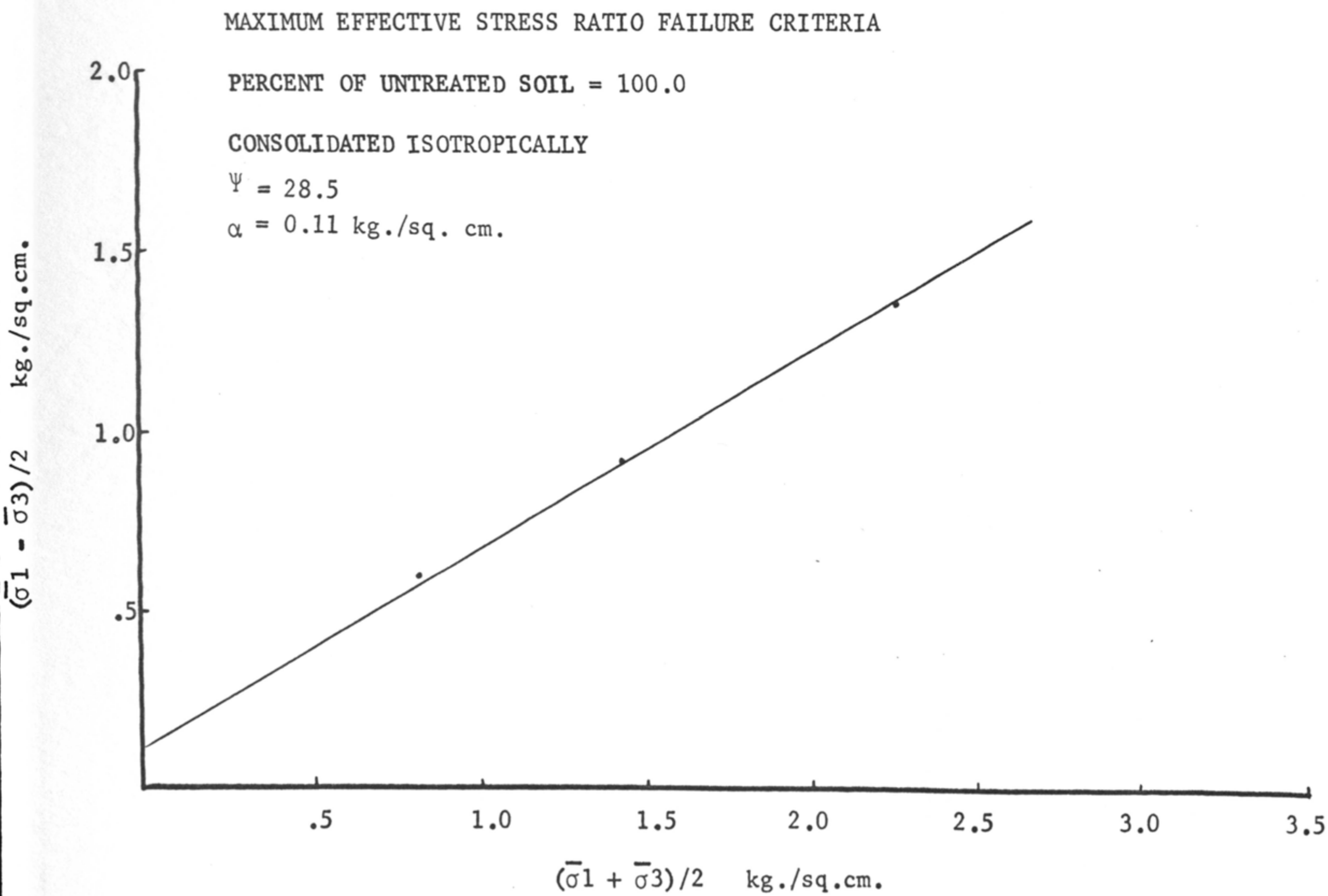


FIG.4b SUMMARY OF SHEAR TEST DATA

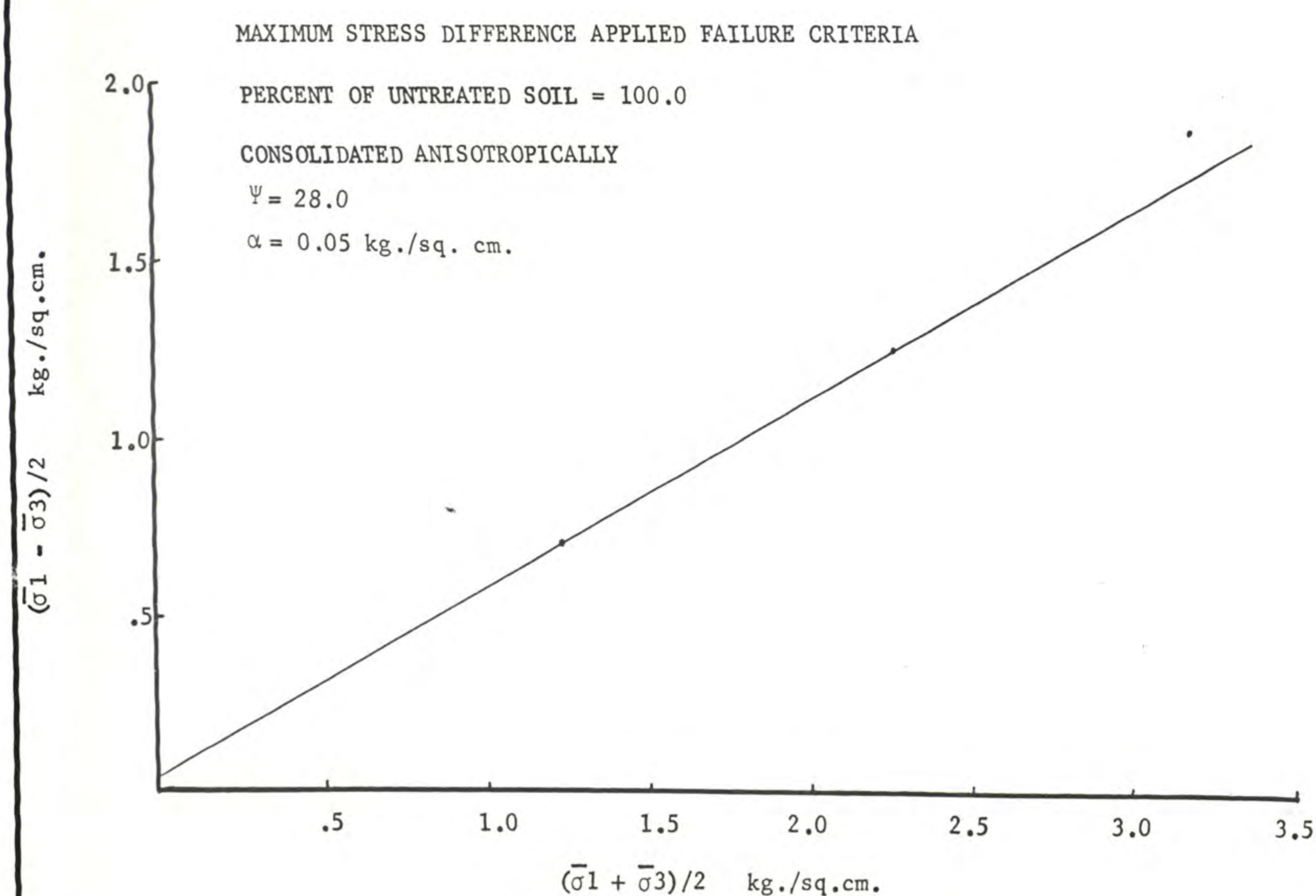


FIG.4c SUMMARY OF SHEAR TEST DATA

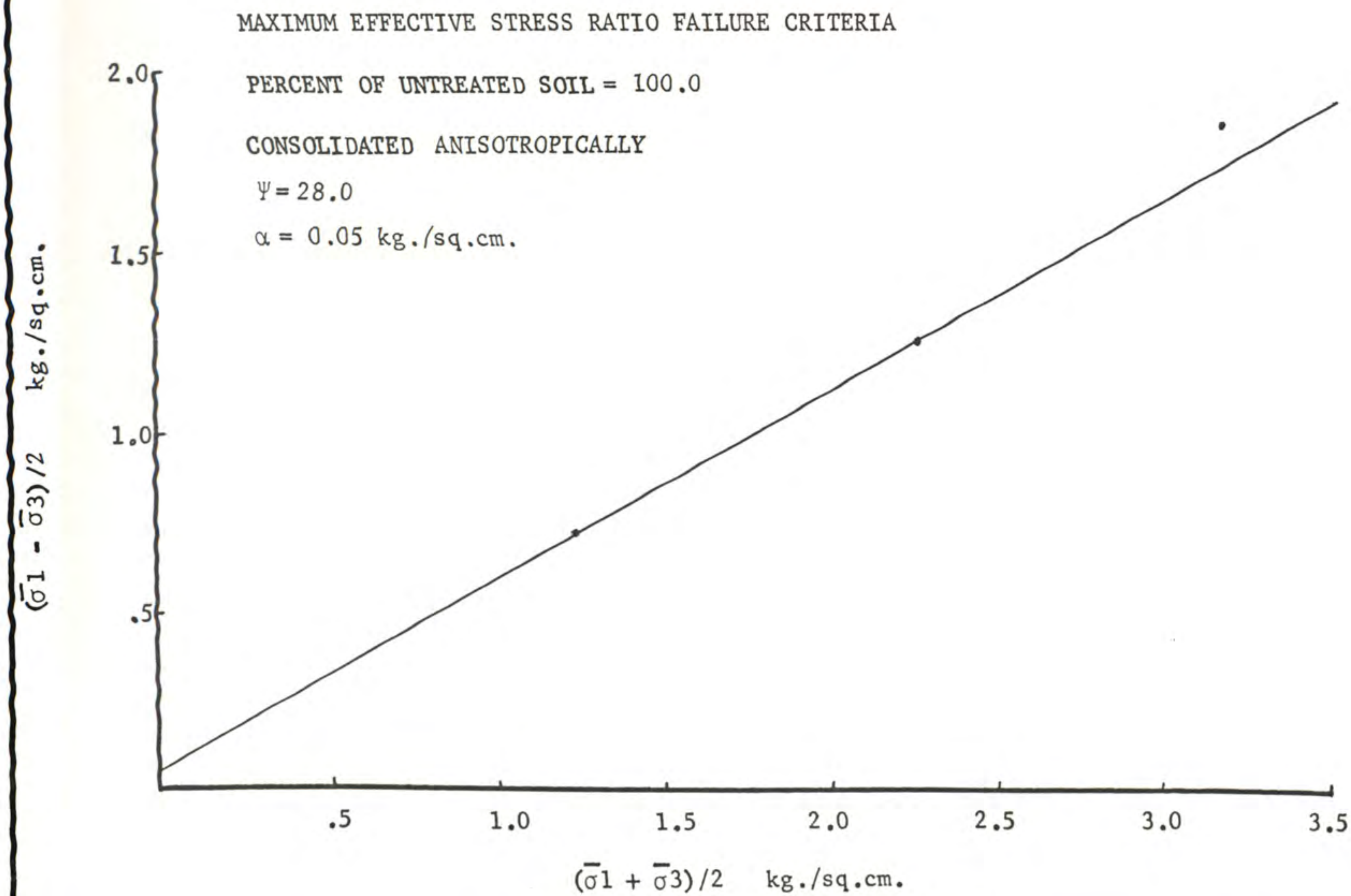


FIG.4d SUMMARY OF SHEAR TEST DATA

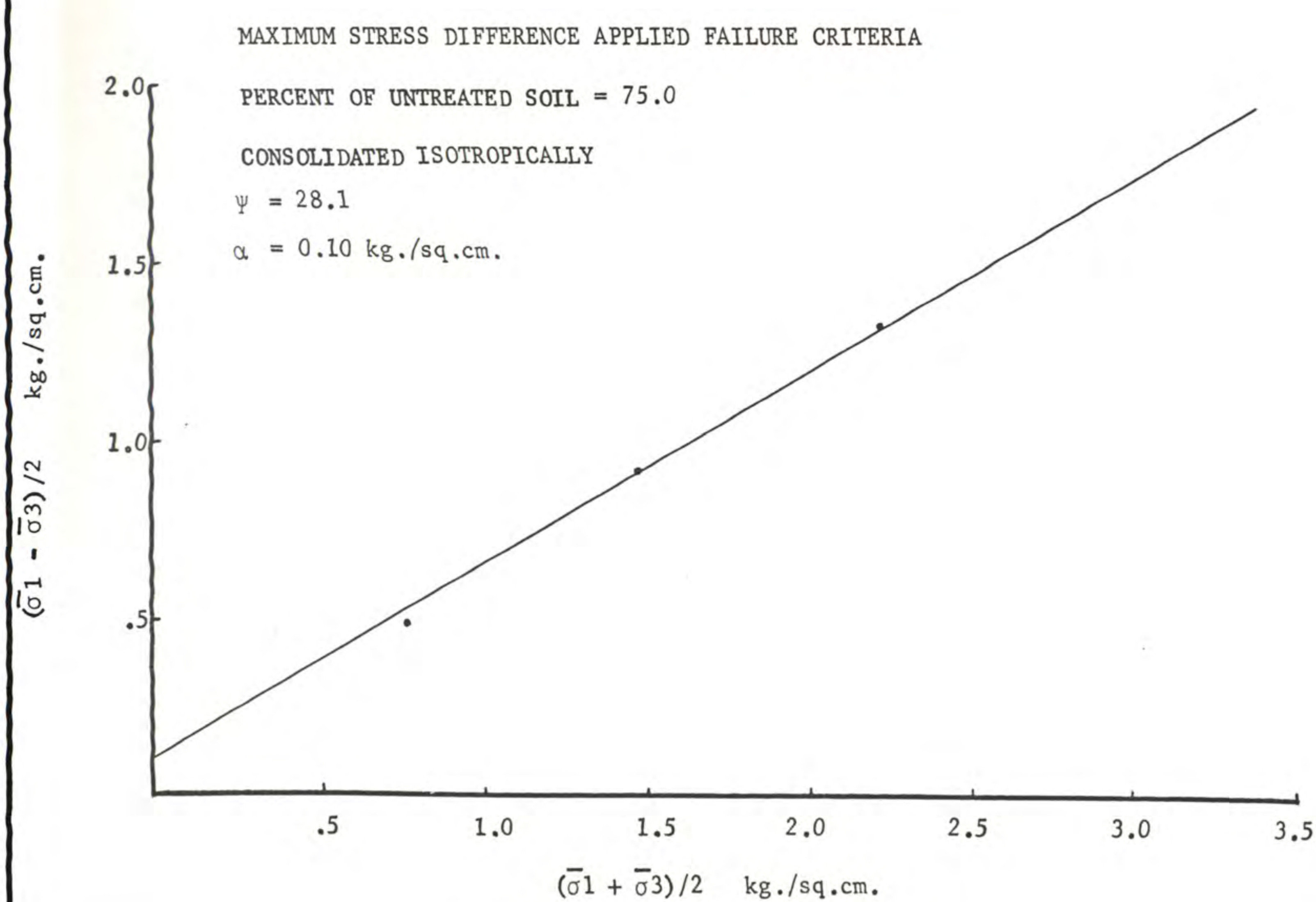


FIG.4e SUMMARY OF SHEAR TEST DATA

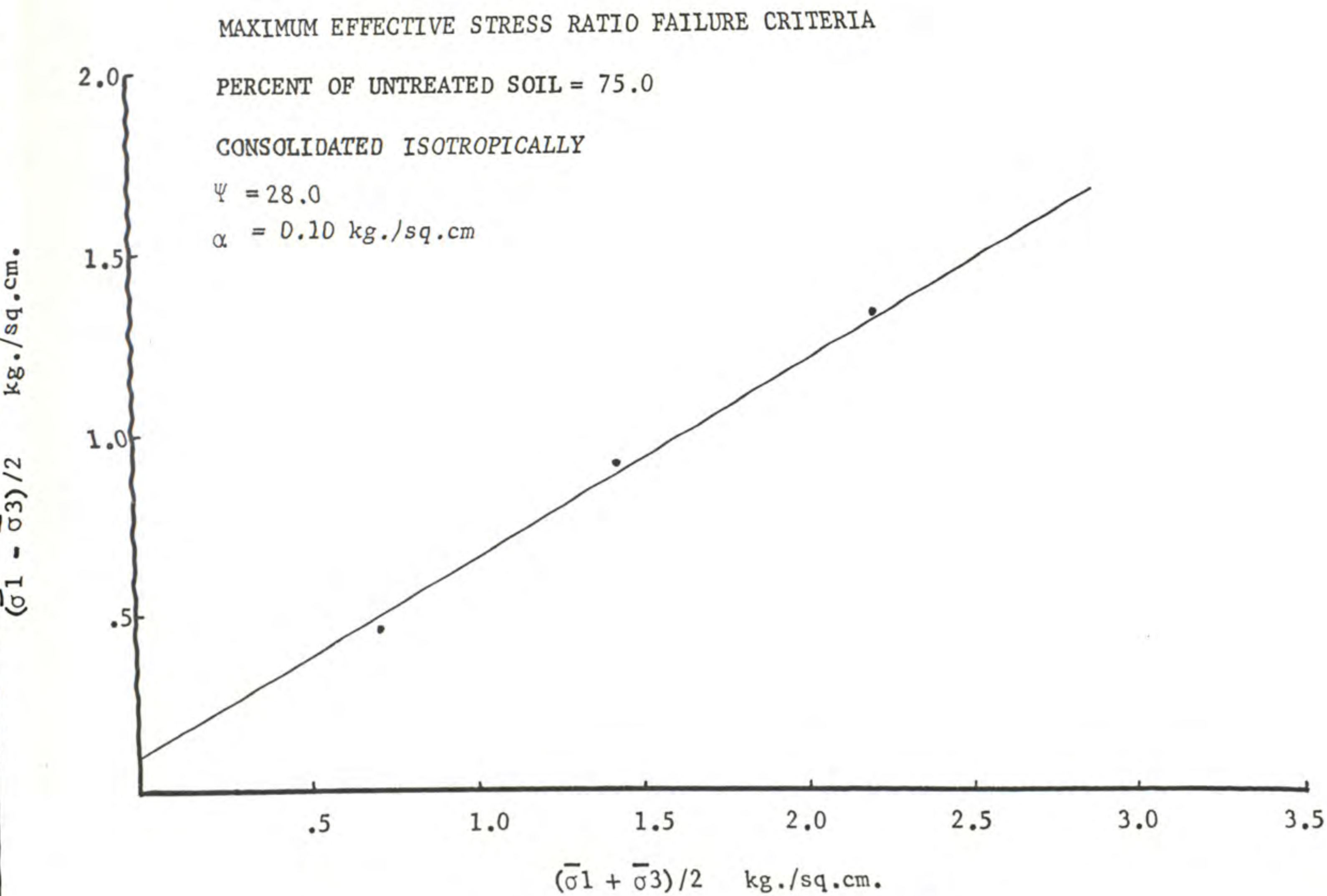


FIG.4f SUMMARY OF SHEAR TEST DATA

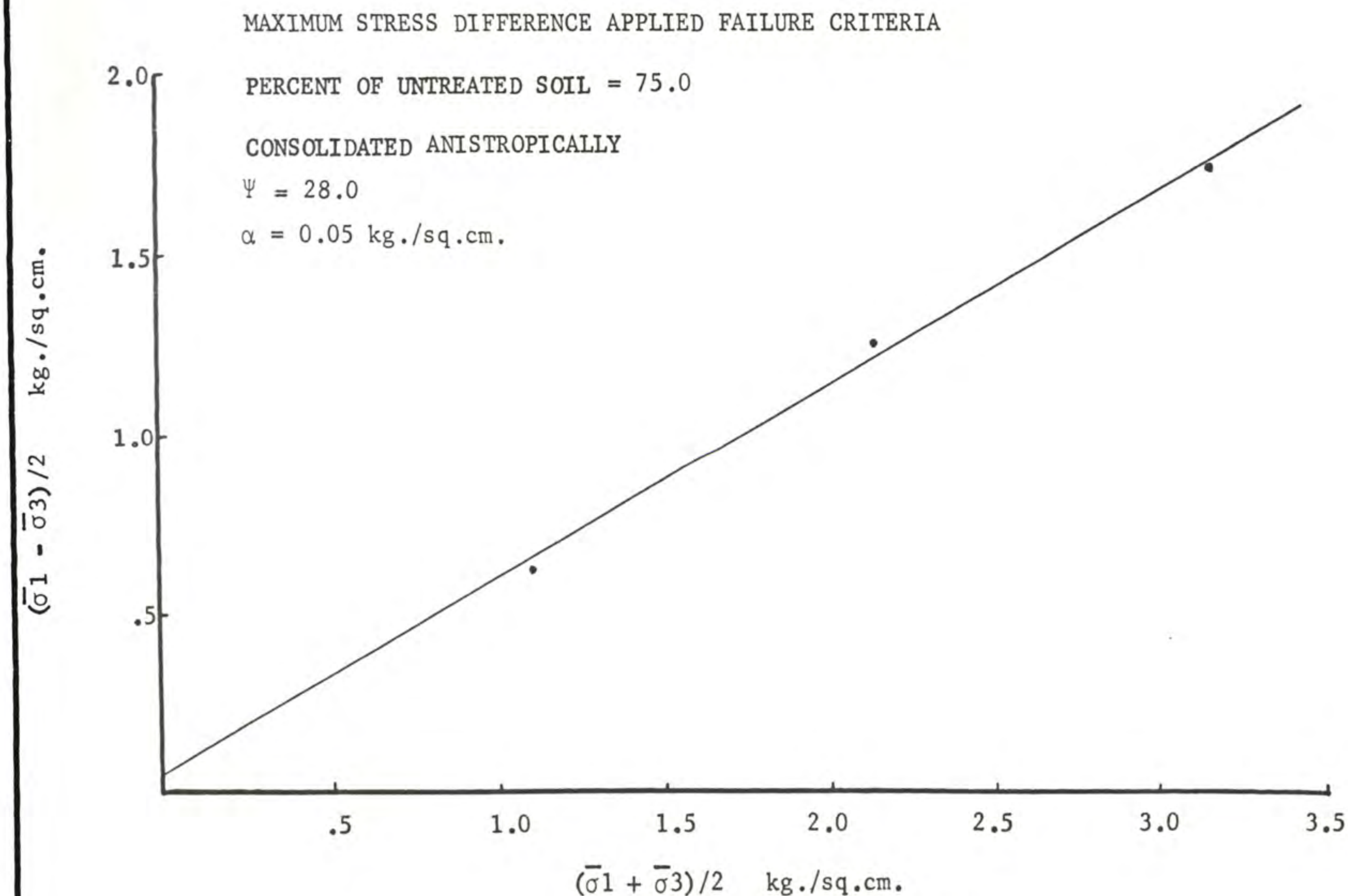


FIG.4g SUMMARY OF SHEAR TEST DATA

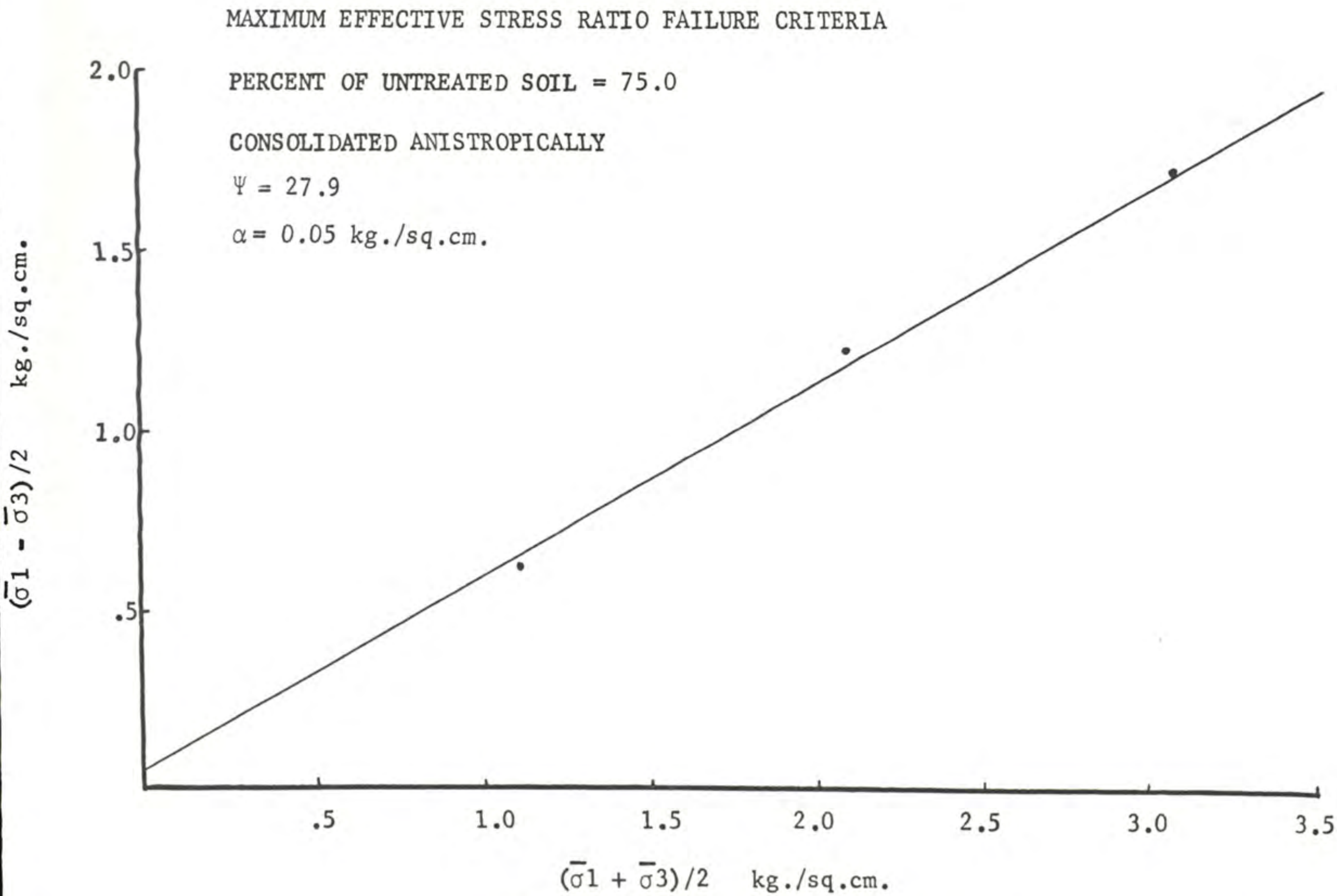


FIG. 4h SUMMARY OF SHEAR TEST DATA

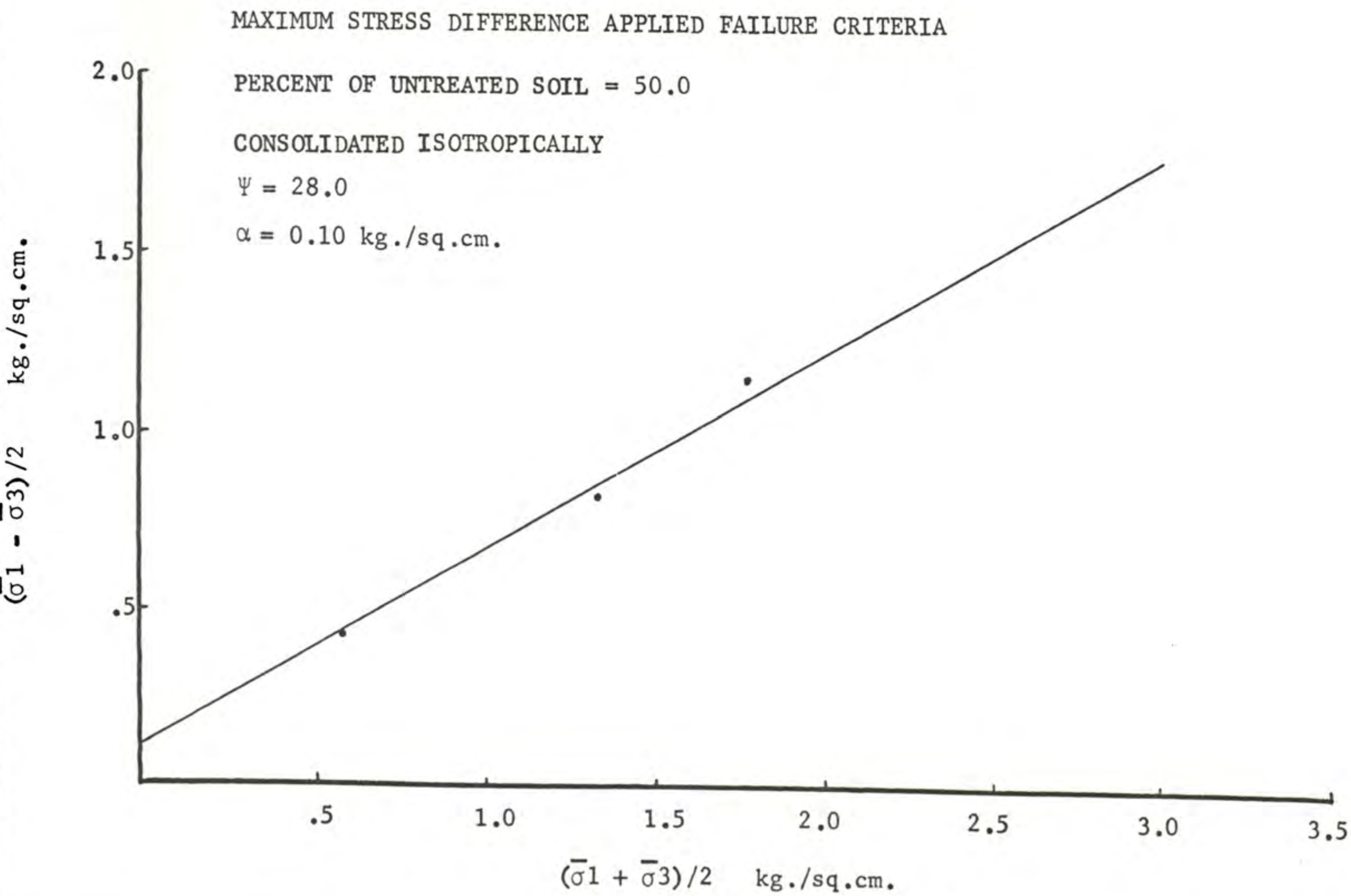


FIG.4i SUMMARY OF SHEAR TEST DATA

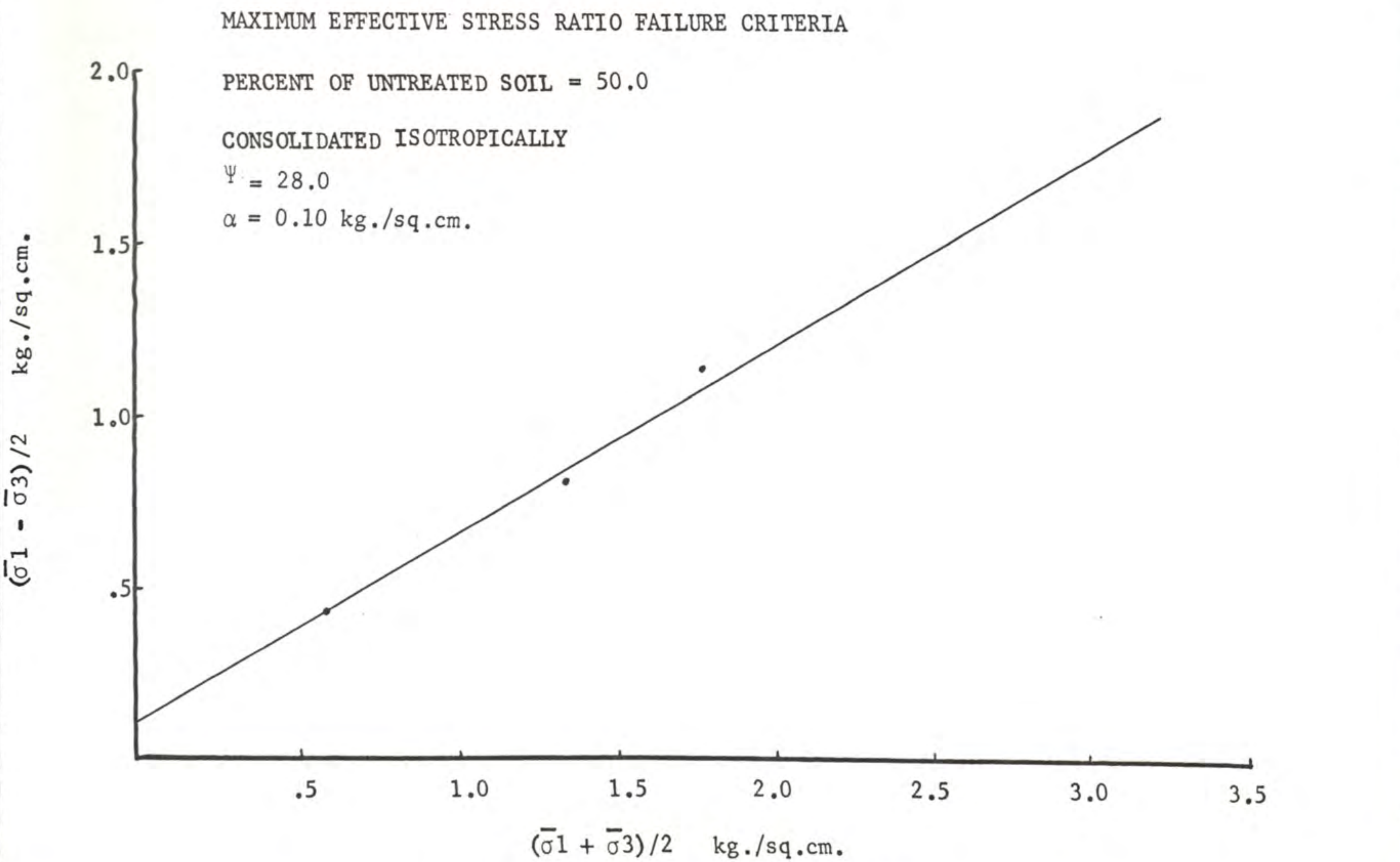


FIG.4j SUMMARY OF SHEAR TEST DATA

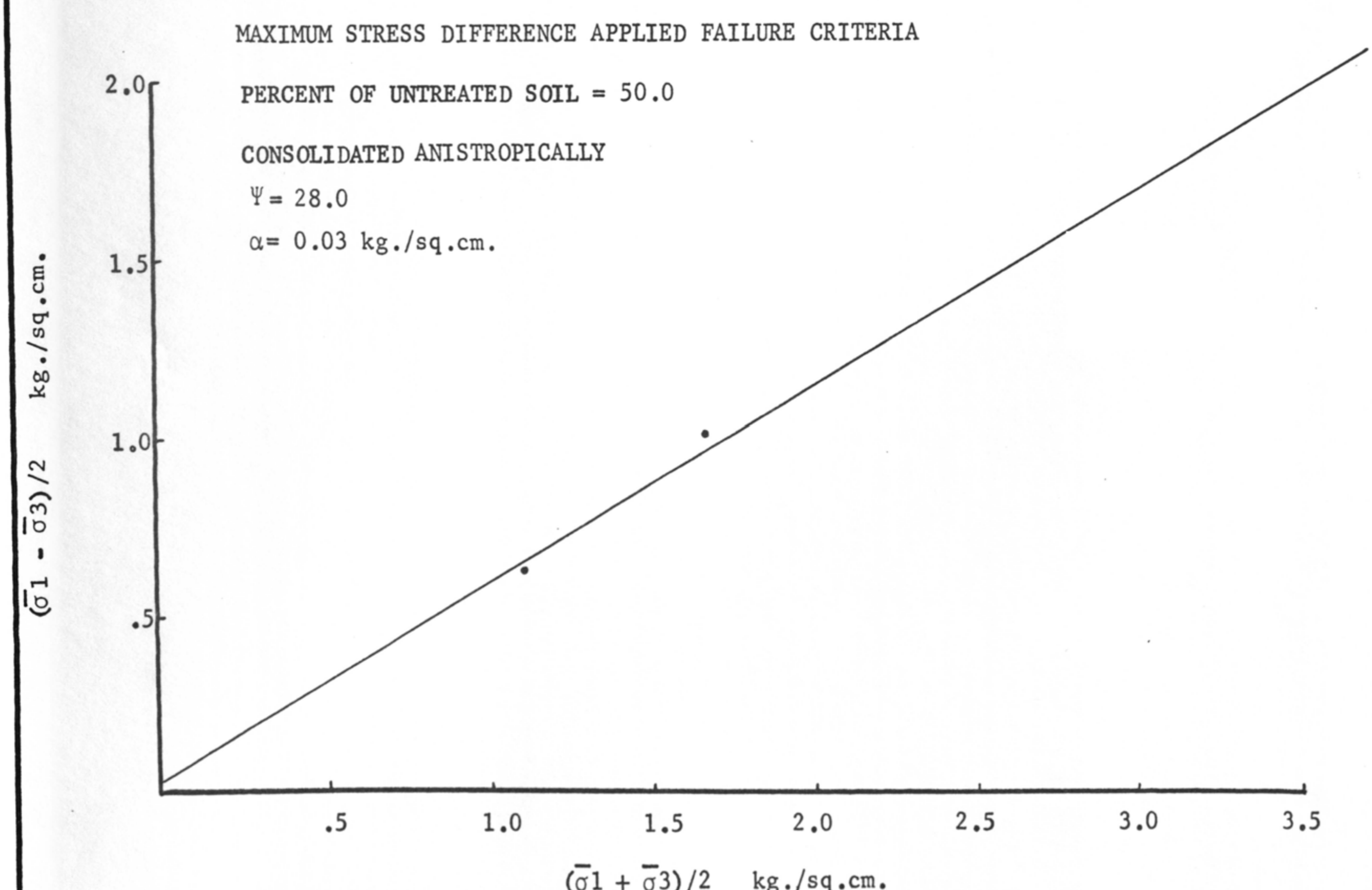


FIG. 4k SUMMARY OF SHEAR TEST DATA

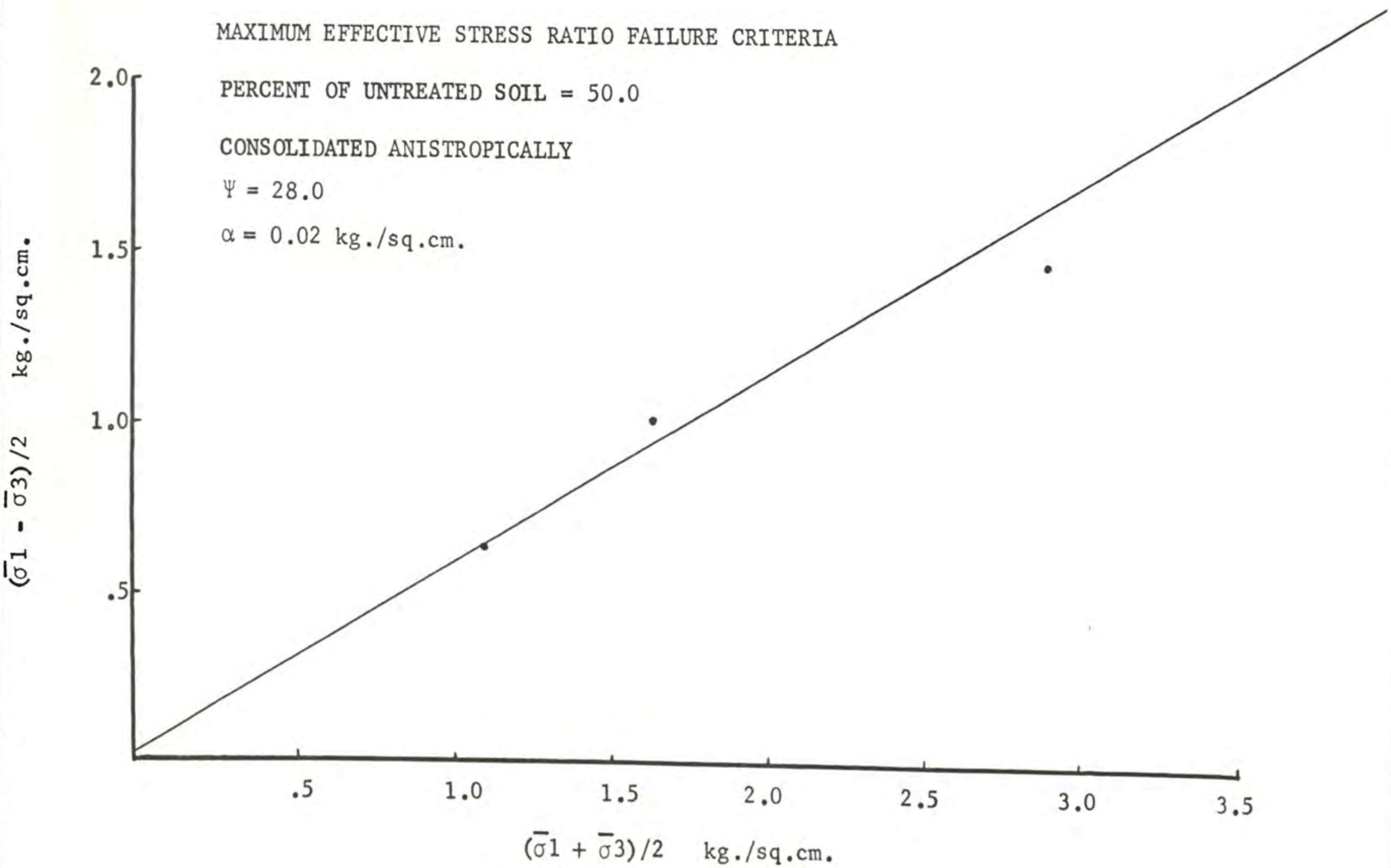


FIG.41 SUMMARY OF SHEAR TEST DATA

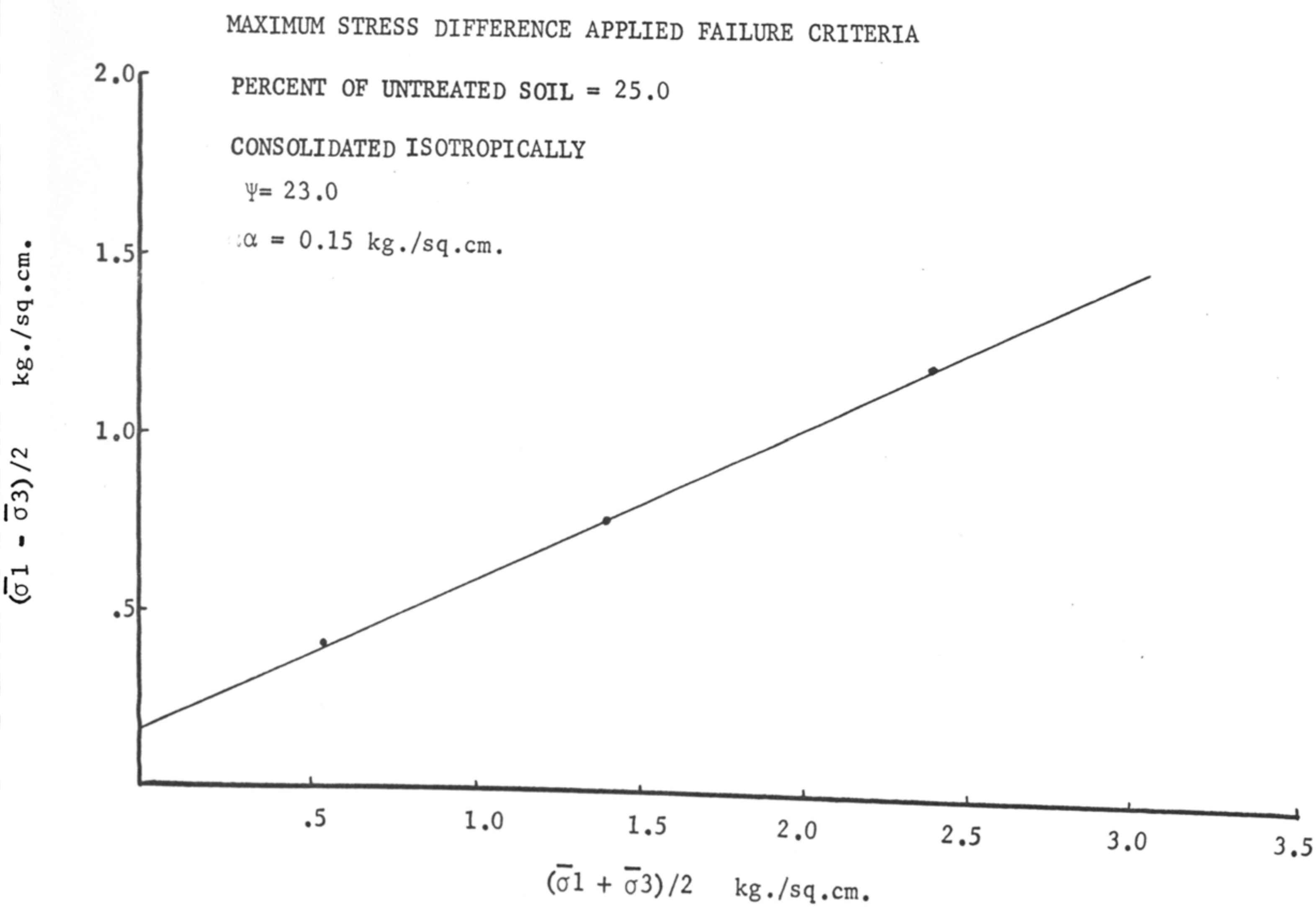


FIG.4m SUMMARY OF SHEAR TEST DATA

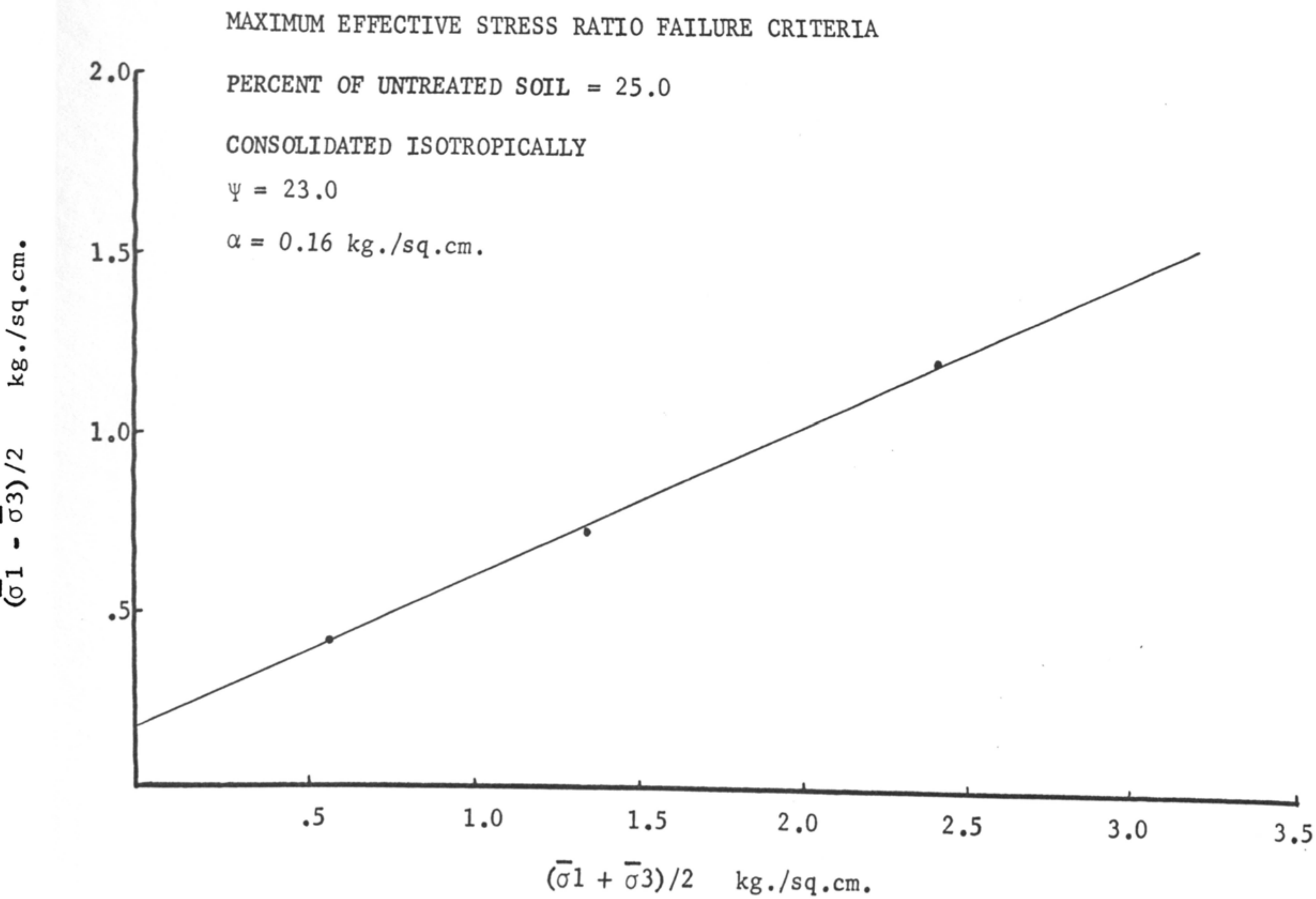


FIG.4n SUMMARY OF SHEAR TEST DATA

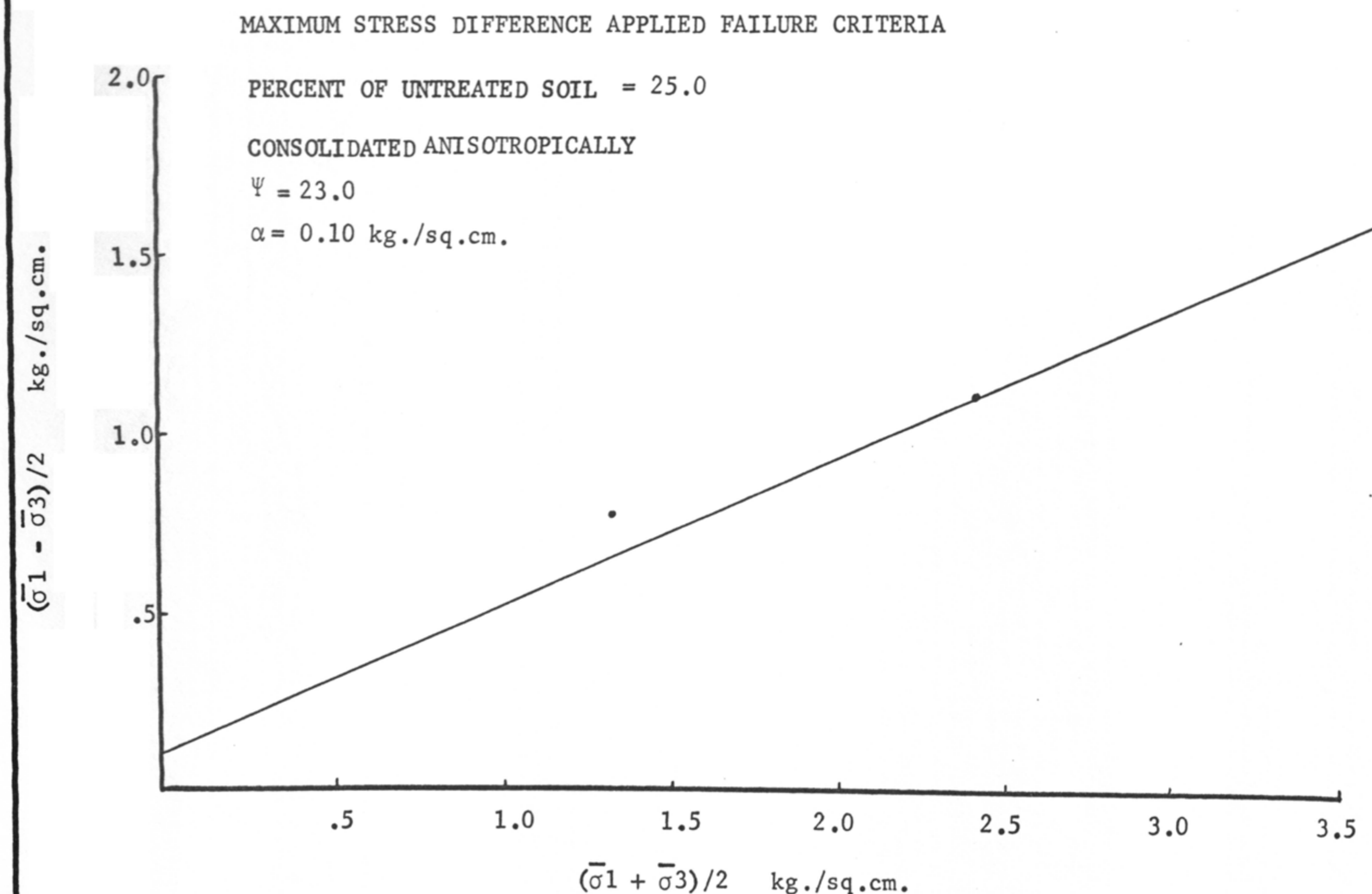


FIG.40 SUMMARY OF SHEAR TEST DATA

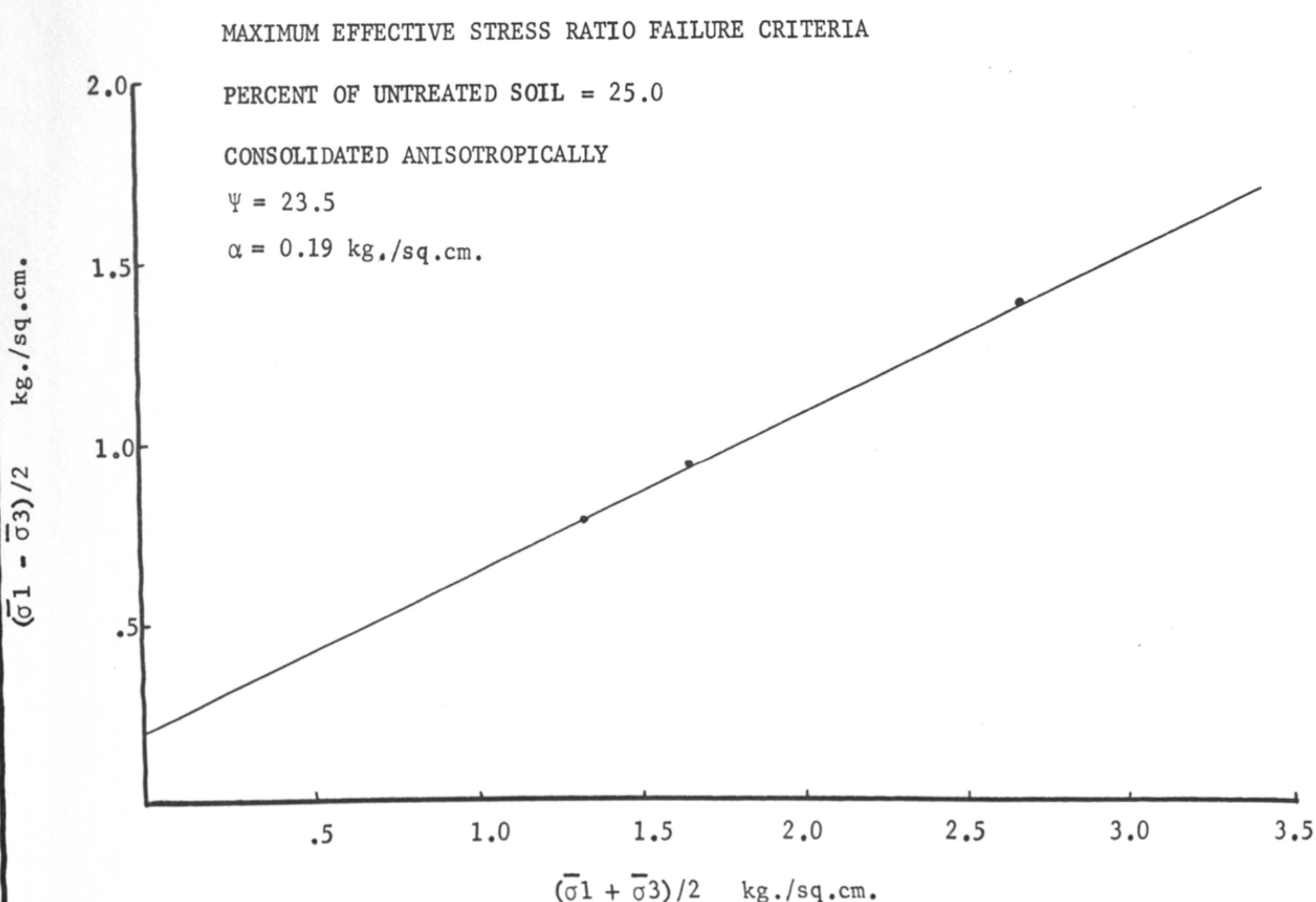


FIG.4p SUMMARY OF SHEAR TEST DATA

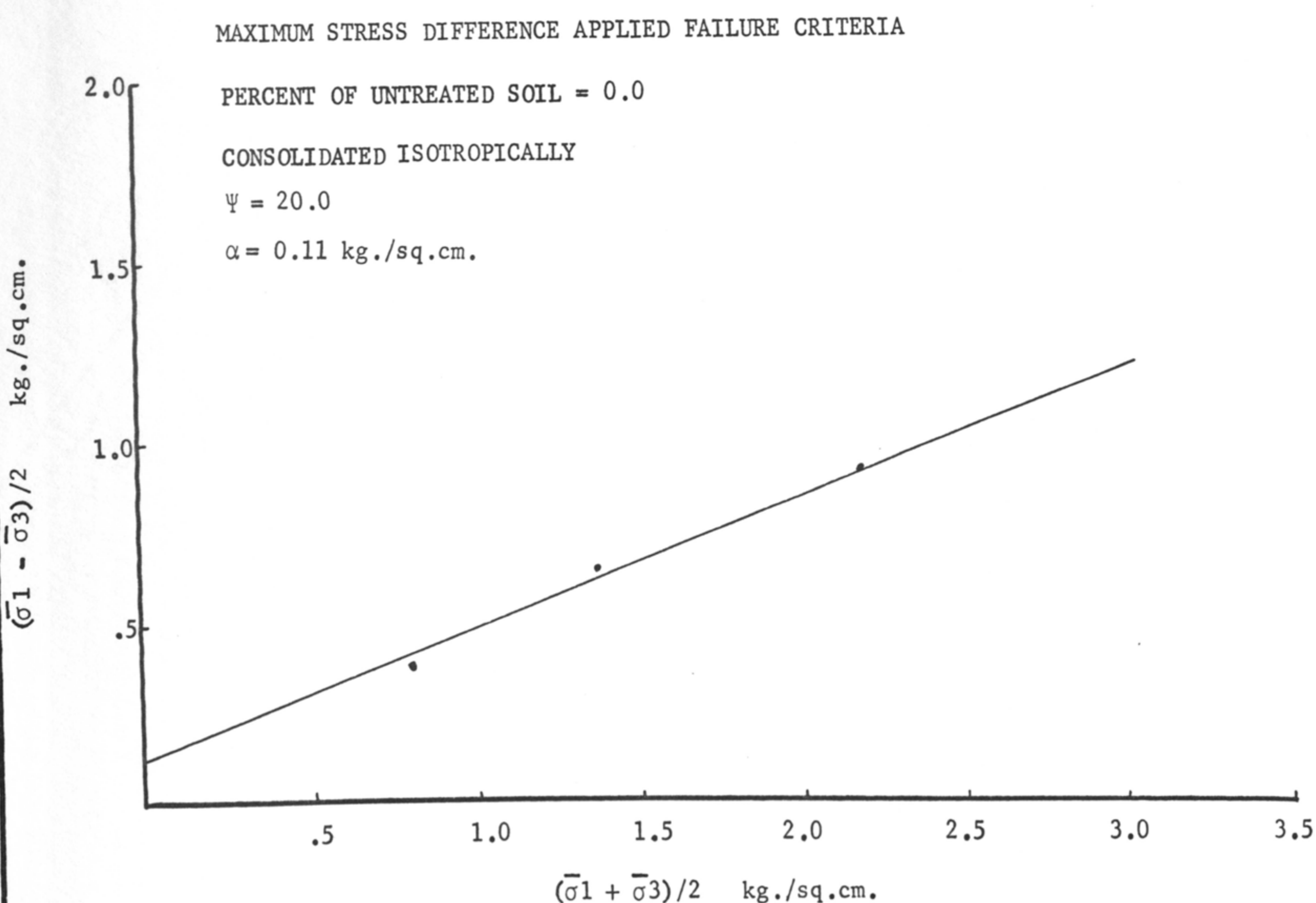


FIG. 4q SUMMARY OF SHEAR TEST DATA

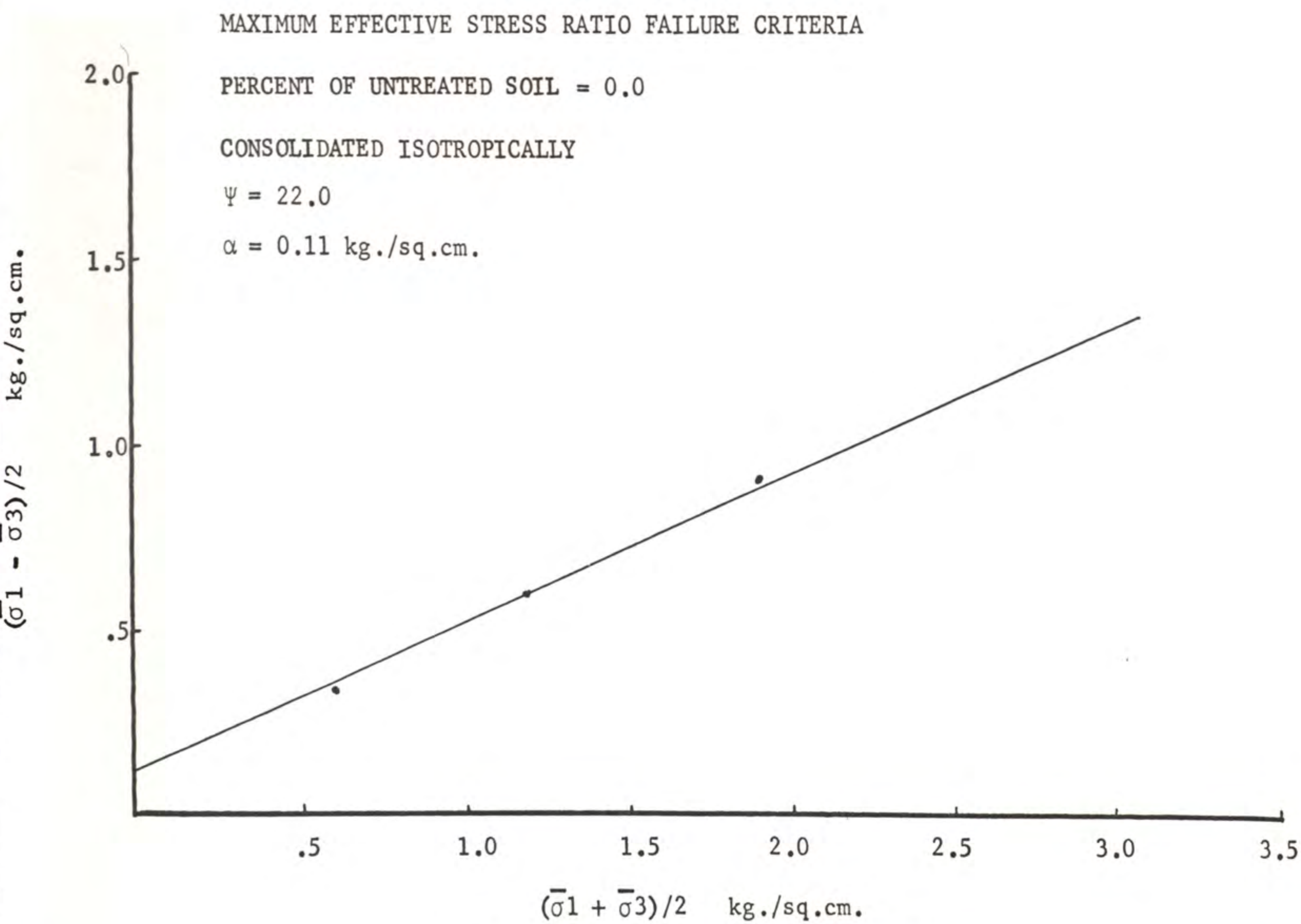


FIG.4r SUMMARY OF SHEAR TEST DATA

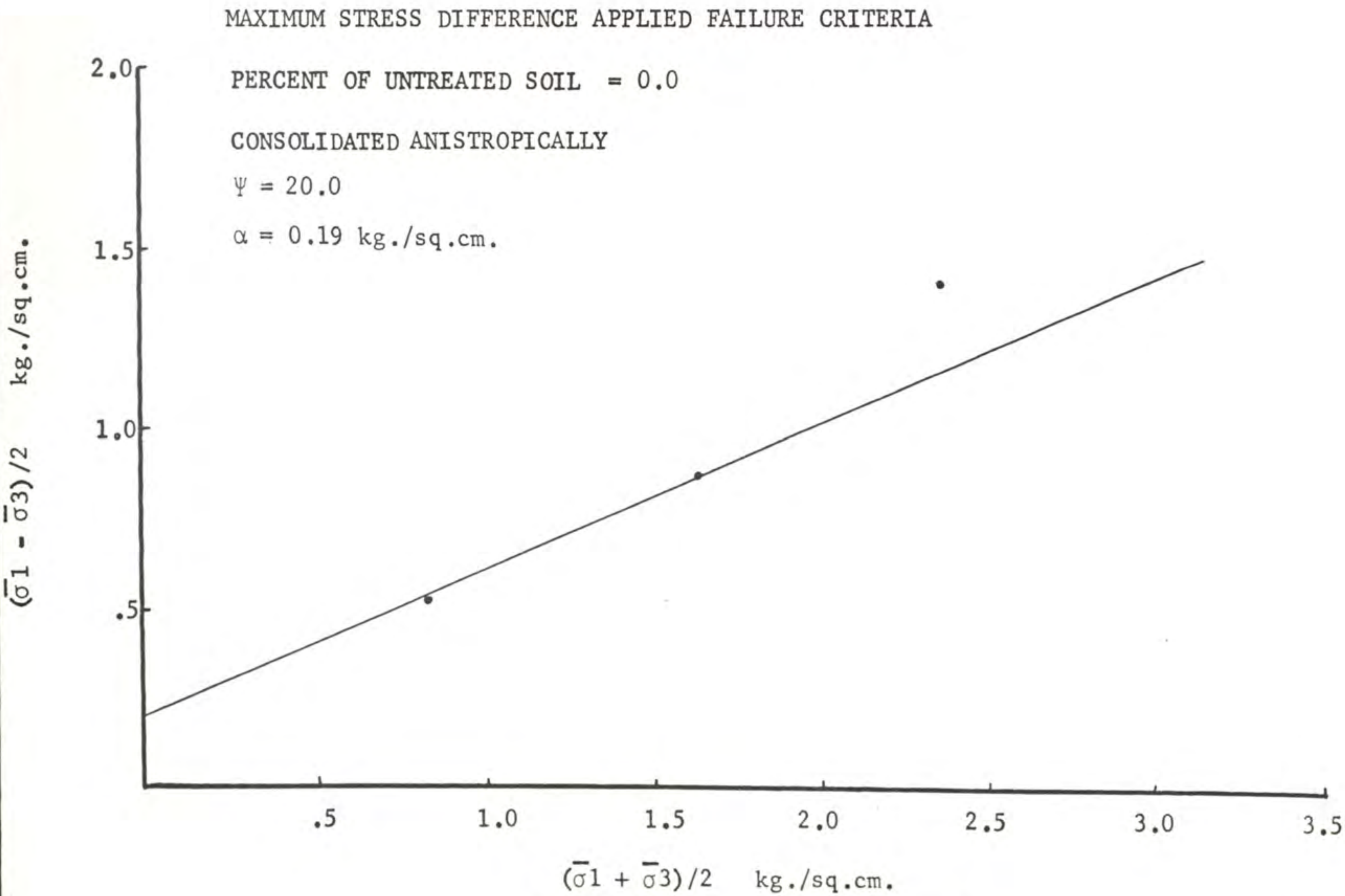


FIG.4s SUMMARY OF SHEAR TEST DATA

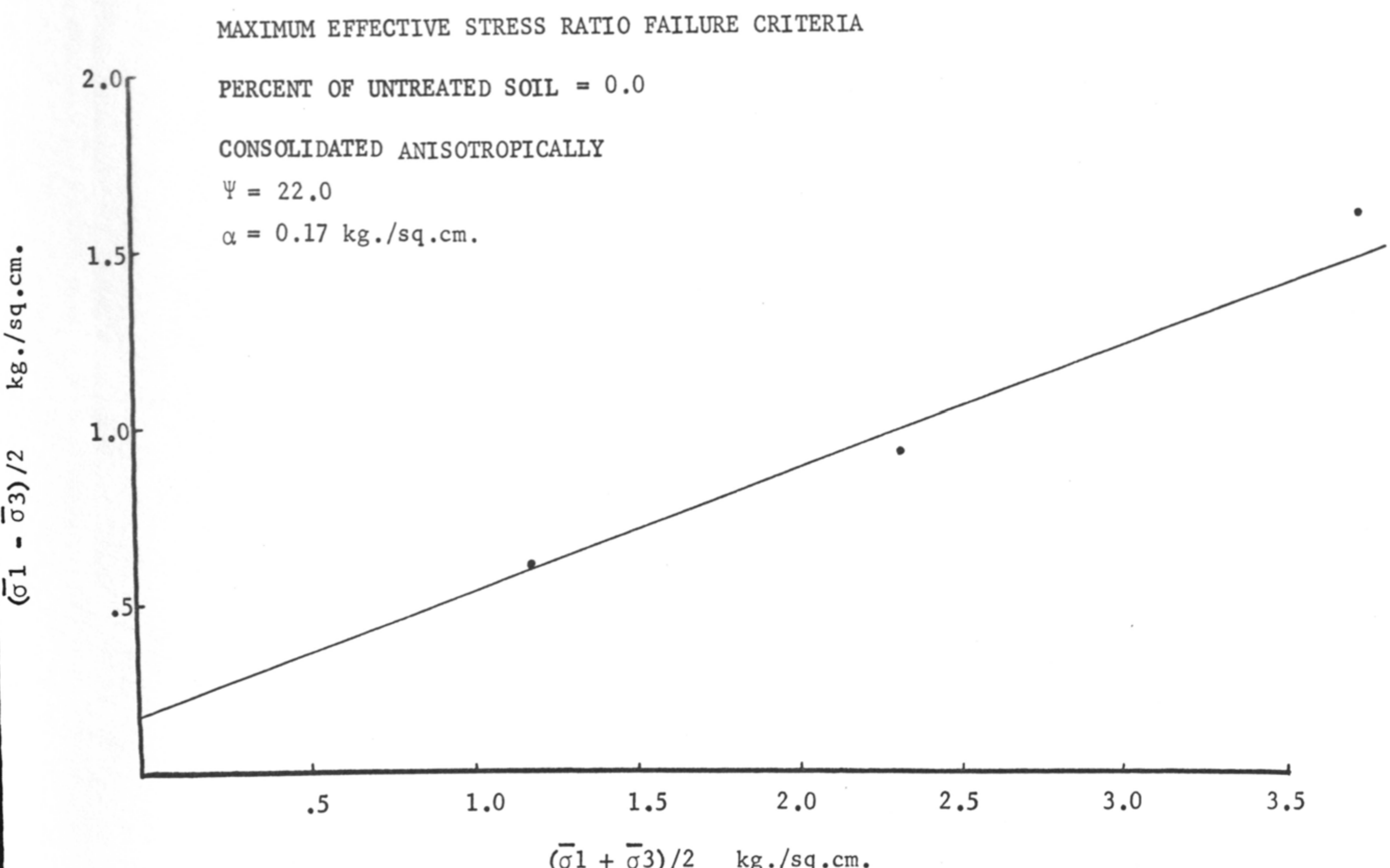
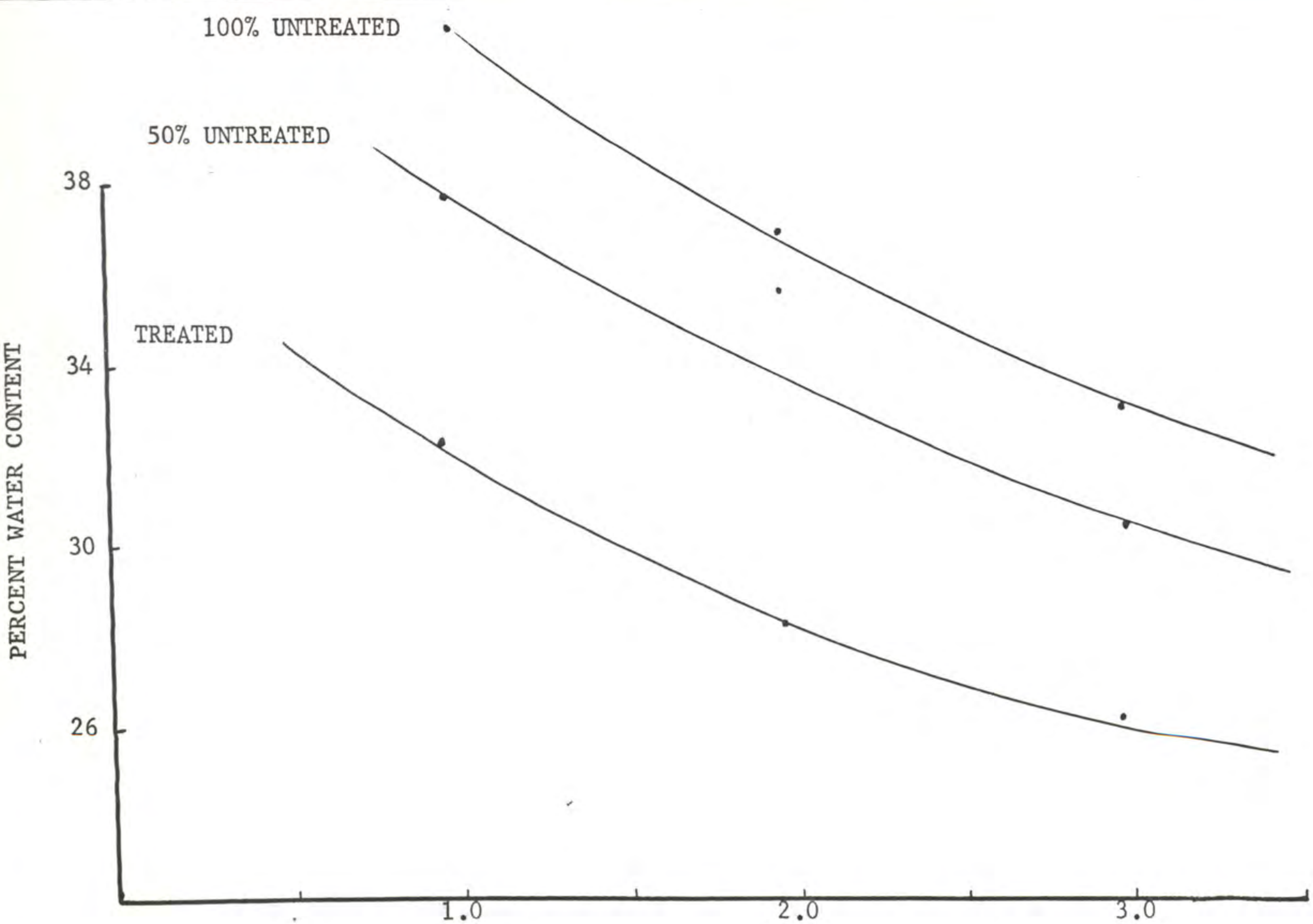


FIG. 4t SUMMARY OF SHEAR TEST DATA

is most pronounced when the quantity of the untreated soil (organic) is between 25 percent and 50 percent of the sample.

3. Water Contents:

In figures 5a and 5b water content is plotted against lateral consolidation pressure for the treated, untreated, and 50 percent untreated soil. Samples consolidated isotropically and consolidated anisotropically are plotted separately, and it is observed that in both states of consolidation the treated soil has a lower water content than the untreated soil under the same confining pressure and the same period of consolidation. Even though the treated soil undergoes a higher volume change and has a lower volume content and void ratio, it still requires less stress difference applied to failure than the untreated soil. The relationship between the water content and the maximum stress difference is presented in figure 6. It is observed that the unique relationship between the water content and the maximum stress difference does exist for the untreated and different mixtures of treated and untreated soils (Henkel, 1960). Both isotropically and anisotropically consolidated samples follow this relationship. However, the tests on the treated soil indicated two separate relationships for the isotropically consolidated samples and the anisotropically consolidated samples. This could be due to experimental errors; however, it is possible that the hydrogen peroxide treatment of the soil could have some effect on the



LATERAL CONSOLIDATION PRESSURE kg./sq.cm.

Fig. 5a WATER CONTENT VS. LATERAL CONSOLIDATION PRESSURE--ISOTROPICALLY CONSOLIDATED SAMPLES

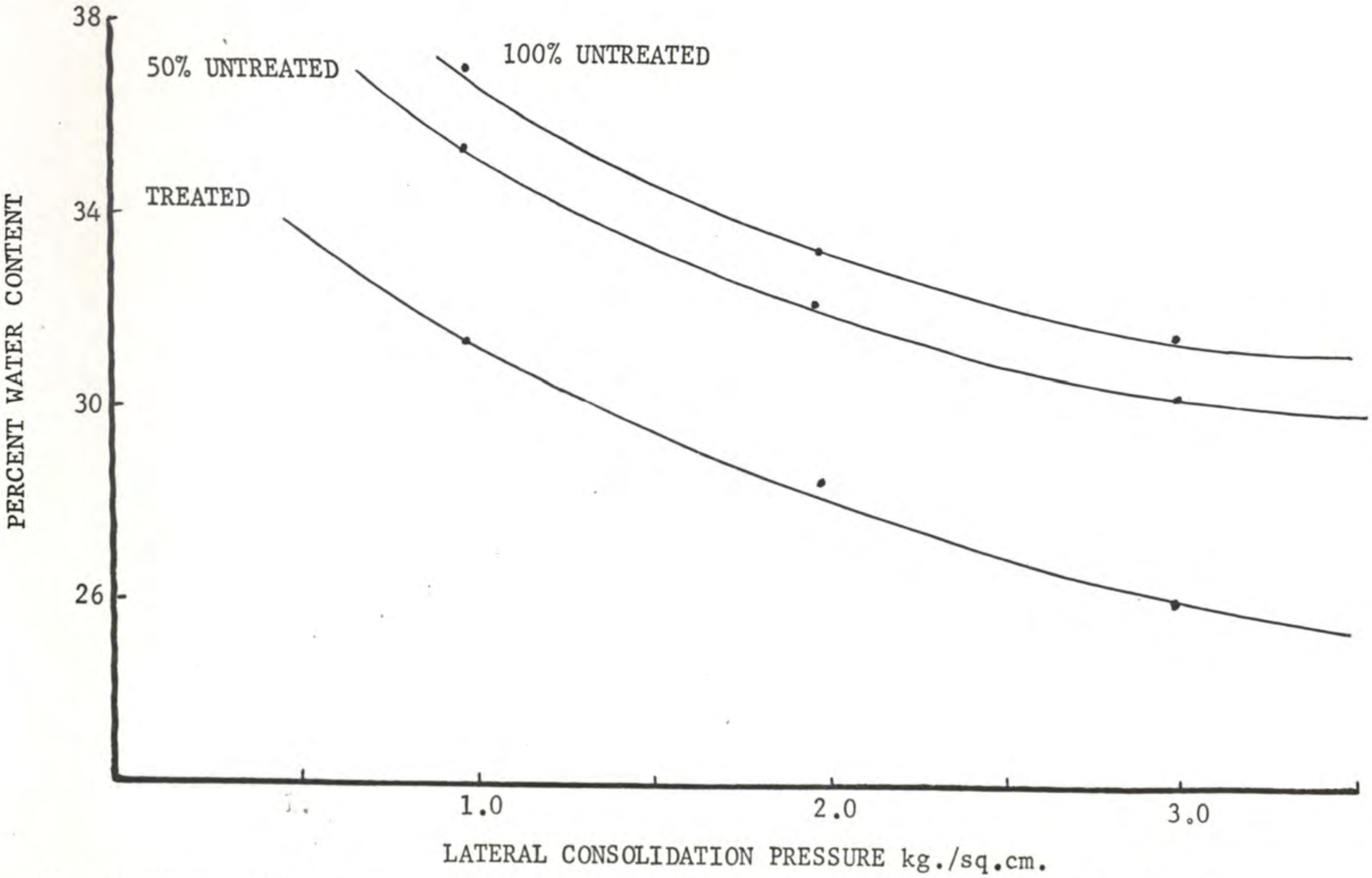


Fig. 5b WATER CONTENT VS. LATERAL CONSOLIDATION PRESSURE--ANISOTROPICALLY CONSOLIDATED SAMPLES

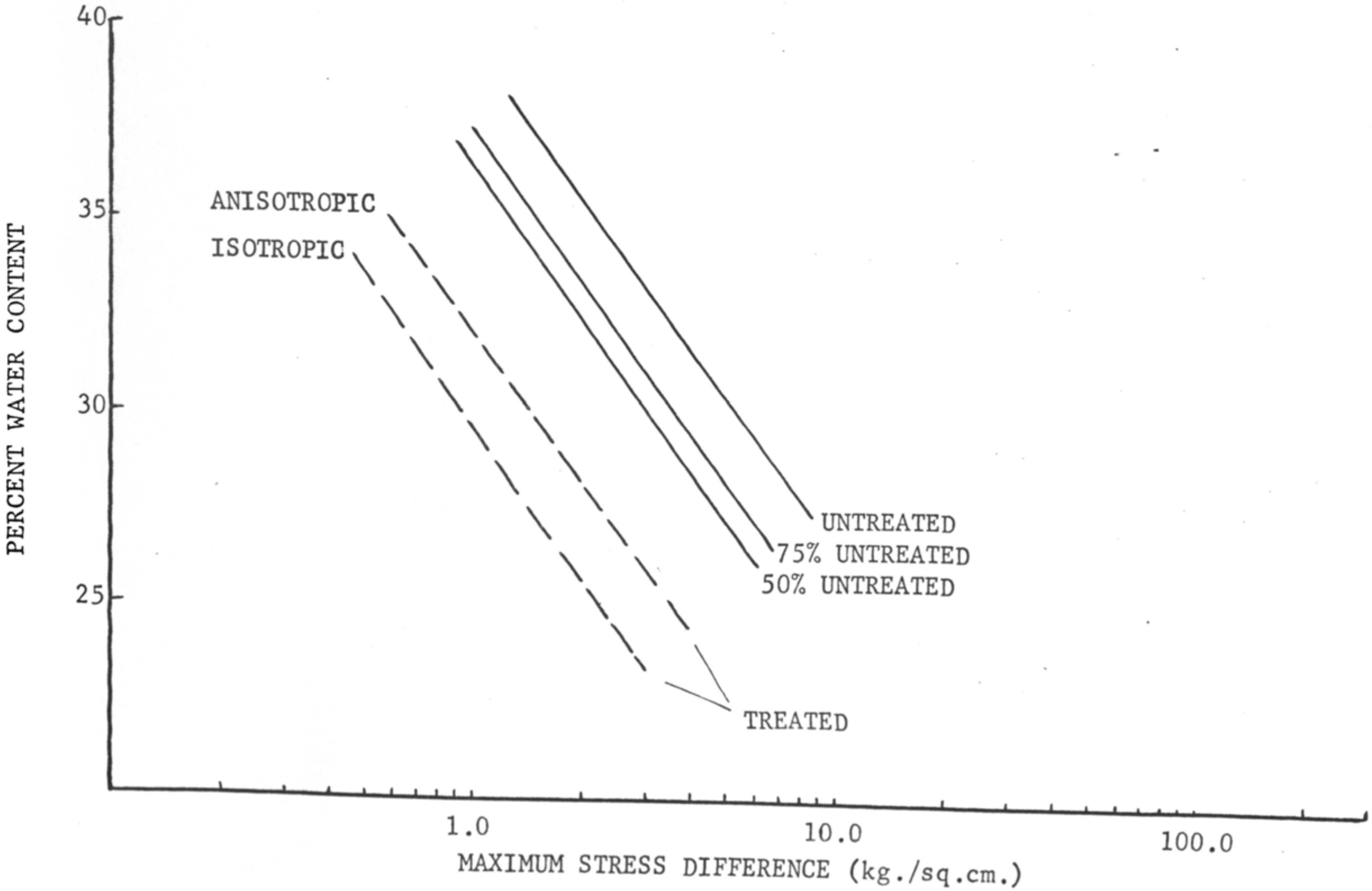


FIG. 6 WATER CONTENT VERSUS MAXIMUM STRESS DIFFERENCE

actual grain structures of the soil particles. To recognize the actual reason for this phenomenon requires further investigation.

C. Pore Pressure Behavior

For the isotropically consolidated samples, test results show that for the untreated soil, pore water pressure developed due to shearing process never exceeds the stress applied to cause failure, i.e. the Skempton's $\underline{A}$ coefficient (Skempton, 1954) is always less than 1.0 for the untreated soil. However, the pore pressure actually exceeds the axial pressure difference applied for the treated soil. Generally, as the amount of carbon increases the maximum value of the $\underline{A}$ coefficient during the shearing process decreases. The values of $\underline{A}$ coefficient at failure are presented in Appendix 2, and designated as $\underline{A}_f$. It should be noted that the values of $\underline{A}_f$ are not the same as maximum values of $\underline{A}$ coefficient. The effect of carbon content on the values of $\underline{A}$ coefficient may be observed on the $\underline{A}_f$ values when the failure criteria is the maximum stress ratio. The $\underline{A}_f$ values of the treated soil do not reflect the effect of carbon when the failure criteria is the maximum stress difference because the maximum stress difference occurs at smaller strains and the time for the pore pressure to develop is not sufficient.

D. Discussion

In general it was found that the presence of organic matter in the soil apparently causes a different structure for the organic soil than for the same soil without the organic matter. The effect of organic matter on the structure of the soil could be to create a bond between the particles or to actually change the structural configuration of the particles. The presence of bonding between the organic soil particles can be substantiated by the laboratory results. During the sample consolidation, the bonded structure tends to hold the water and prevent its movement out of the sample. The treated soil, however, does not have this extra bonding. This could explain the lower water content in the treated soil. The phenomenon can also be caused by different structural configurations. A flocculated structure tends to trap more water between the particles than a dispersed structure. It can also cause more interlocking between the particles which could be the reason that the organic soil displays a higher strength in terms of the effective internal friction angle.

The variations of A coefficients for the treated and the untreated soils can also be explained by the different structural configuration. Seed, Mitchell, and Chan (1960) substantiate this by observing that the dispersed structure develop higher pore pressures during shear than the flocculated structure. This is in agreement with the test results obtained, i.e. the treated soil develops higher pore pressures than the untreated soil. Again, the existence of bonding between the particles can also display this behavior.

Figure 6 is an illustration of the relationship between the undrained strength of the treated (inorganic) and the untreated (organic) soils and the water content. It is shown that the undrained strength of the untreated soil is approximately fivefold greater than the treated soil with the same water content. It also indicates that for a given strength the untreated (organic) soil can hold a higher water content and still maintain its strength. This behavior indicates a much stronger structure for the untreated soil than for the treated soil at a given volume. The phenomenon can also be caused by the structural configuration or bonding between the particles. Nevertheless, flocculated structures tend to display a high strength at small strains and decrease in strength as strain increases. Figures 3a through 3d present typical stress-strain relationship for the treated and the untreated soils. It can be seen that the gain in strength for the untreated soil as strain increases tends to disagree with the structural configuration hypothesis. In general it appears that the existence of bonding between the particles provides a better hypothesis. The stress-strain relationship, however, seems to indicate that the bonding is one of compressible glue-type nature rather than a brittle cement-like bonding.

CHAPTER V

CONCLUSIONS

The presence of organic matter has a definite effect on the shear strength characteristics of the cohesive soil. The treated (inorganic) soil displays a lower strength based on the effective internal friction angle. The gain in strength for the organic soil is believed to be due to the presence of some bonding between the particles, the exact nature of which is not certain.

The untreated soil has a smaller modulus of deformation. The treated soil, in general, fails at lower strains than the untreated soil regardless of the condition of consolidation, when the failure criteria is the maximum stress difference.

The effective internal friction angle of the soil is independent of the state of consolidation.

The treated soil undergoes more volume change under the same consolidation pressure than the untreated soil.

There is a unique relationship between the water content and the undrained strength of the soil for the untreated soil and mixtures of the treated and untreated soils, regardless of the state of consolidation. The treated soil presents two separate relationships for the isotropic and anisotropic states of consolidation. The untreated (organic) soil has an undrained strength approximately 5 times larger than the undrained strength of the treated soil with the same water content. Intermediate mixtures of organic-inorganic soils display intermediate strengths approximately according to their carbon contents.

A coefficients developed due to shearing process are higher for the treated soil than that for the untreated soil.

CHAPTER VI

RECOMMENDATIONS FOR FUTURE RESEARCH

During the course of this project, some subjects seemed to need further investigation, and some were out of the scope of this investigation but worthwhile for future research. Among these subjects are as follows:

A. The Slower Rate of Strain - It was found that when the treated soil was consolidated anisotropically the deviator stress reached its maximum at very low strains. This might have affected the measurement of pore pressure developed due to shearing process. It seems some tests should be performed at lower strain rates and results be compared and stress paths studied.

B. Strength versus Degree of Consolidation - Triaxial consolidation can be performed with measurement of pore water dissipation. Triaxial shear tests could be performed on samples consolidated to various degrees and results be studied for the gain in strength as degree of consolidation increases. The above can be performed on treated and untreated organic soils and the effect of carbon content can be studied at different degrees of consolidation.

C. Anisotropic Consolidation - Anisotropically consolidated samples for this project were consolidated so that the ratio of vertical to lateral consolidation pressure were kept constant during the process. However, samples can be consolidated isotropically, and then additional vertical consolidation pressure be applied to desired magnitude. Results can be studied for the effect of stress history of the sample.

D. Effect of Treatment - More isotropically consolidated and anisotropically consolidated triaxial shear tests should be performed on the treated soil to investigate the phenomenon presented in figure 6. Whether the treatment of the soil actually changes the grain structure or not could be very useful for further research.

APPENDIX 1

List of Symbols

$\underline{A}_f$	Skempton's $\underline{A}$ coefficient at failure
C	Cohesion intercept
e	Void ratio
u	Pore water pressure
u_f	Pore water pressure at failure
w	Water content
α	Cohesion intercept for modified mohr diagrams
ε_f	Strain at failure
ψ	Friction angle for modified mohr diagram
Φ'	Friction angle between the particles
$\bar{\sigma}1$	Vertical effective pressure
$\bar{\sigma}3$	Lateral effective pressure
$\bar{\sigma}1c$	Vertical Consolidation pressure
$\bar{\sigma}3c$	Lateral Consolidation pressure

APPENDIX 2

Sample of Computer Program and Test Data

```
      DIMENSION SDIA(50),SLENG(50),DVOL(50),SLENGF(50),ALDR(50,50),
     1ADDR(50,50),ATDR(50,50),P1(50),P3(50),WW(50,3),DW(50,3),CW(50,3),
     2CM(50,3),CPER(50),NUMREA(50),EV(50,3),SG(50)
      READ(1,13)NUMSAM
   13 FORMAT(I2)
      DO 100 N=1,NUMSAM
      READ(1,1) SDIA(N),SLENG(N),DVOL(N),SLENGF(N),P1(N),P3(N),CPER(N),S
     1G(N),NUMREA(N)
    1 FORMAT(2F3.2,F4.2,3F3.2,F3.0,F3.2,I3)
      NUMBER=NUMREA(N)
      READ(1,2) (ADDR(N,I),ALDR(N,I),ATDR(N,I),I=1,NUMBER )
    2 FORMAT(F4.0,F4.1,F4.0)
      READ(1,3) (WW(N,L),DW(N,L),CW(N,L),L=1,3)
    3 FORMAT(3F4.2)
      SAREA= (SDIA(N)**2.0)*3.1415/4.0
      VOLUME=(SAREA*SLENG(N))
      COVOL=VOLUME-DVOL(N)
      DEFORM=ADDR(N,NUMBER)*0.002540
      COLENG=SLENGF(N)+DEFORM
      COAREA=COVOL/COLENG
      WRITE(3,4)N
    4 FORMAT('1',14X,'RESULTS OF TRIAXIAL TEST PERFORMED ON 100% SATURAT
     1ED SAMPLE NUMBER'I3)
      ANISOR=  P1(N)/P3(N)
      WRITE(3,5) CPER(N), ANISOR
    5 FORMAT(/20X,'PERCENT OF CARBON SOIL=',F4.0,6X,'ANISOTROPIC RATIO='
     1,F5.2)
      WRITE(3,19) P1(N),P3(N)
   19 FORMAT(/19X,'VERTICAL CONSOLIDATION PRESSURE=',F5.2,6X,'LATERAL CO
     1NSOLIDATION PRESSURE='F5.2)
      WRITE(3,6)
    6 FORMAT(/1X,'STRESS',2X,'STRAIN',3X,'AREA',2X,'PORE T',2X,'PORE D',
     12X,'SIGMA1',2X,'SIGMA3',2X,'SIG1EF',2X,'SIG3EF',2X,'DEVSTR',2X,'ST
     1RESS',2X,'ACOEFF',1X,'DEVST/2P3',1X,'SIG3EF/P3',1X,'V',6X,'H')
      WRITE(3,7)
    7 FORMAT( 1X,'KGSQCM',2X,'PERCEN',3X,'SQCM',2X,'KGSQCM',2X,'KGSQCM',
     12X,'KGSQCM',2X,'KGSQCM',2X,'KGSQCM',2X,'KGSQCM',2X,'KGSQCM',2X,'RA
     1TIO',3X,'VALUE',4X,'VALUE',5X,'VALUE')
      DO 200 I=1,NUMBER
      STRAIN=ADDR(N,I)*0.002540/COLENG
      PERSTR=STRAIN*100.0
      TOTLD=ALDR(N,I)*0.2807
      AREA=COAREA/(1.0-STRAIN)
```

```
      SIGMA1=P1(N)+TOTLD/AREA
      SIGMA3=P3(N)
      UPP=ATDR(N,I)*0.0014
      DUPP=UPP-ATDR(N,1)*0.0014
      SIG1EF=SIGMA1-DUPP
      SIG3EF=SIGMA3-DUPP
      DEVS=SIGMA1-SIGMA3
      STRAT=SIG1EF/SIG3EF
      IF(I.EQ.1) GO TO 33
      ACOEFF=DUPP/(TOTLD/AREA)
      GO TO 22
   33 ACOEFF=0.0
      GO TO 22
   22 STRESS=TOTLD/AREA
      X=SIG3EF/P3(N)
      Y=DEVS/(2.0*P3(N))
      V=(SIG1EF-SIG3EF)/2.0
      H=(SIG1EF+SIG3EF)/2.0
      WRITE(3,8) STRESS,PERSTR,AREA,UPP,DUPP,SIGMA1,SIGMA3,SIG1EF,SIG3EF
     1,DEVS,STRAT,ACOEFF,Y,X,V,H
    8 FORMAT(/1X,F6.3,2X,F6.3,2X,F5.2,1X,F6.3,2X,F6.3,2X,F6.3,2X,F6.3,2X
     1,F6.3,2X,F6.3,2X,F6.3,2X,F6.3,2X,F6.3,3X,F6.3,4X,F6.3,2X,F6.3,4X,
     1F6.3)
  200 CONTINUE
C     MOISTURE CONTENTS AND VOID RATIOS
      WRITE (3,20)
   20 FORMAT('1')
      WRITE(3,9)N
    9 FORMAT(    6X,'MOISTURE CONTENTS OF SAMPLE NO.',I3)
      DO 300 L=1,3
      CM(N,L)=(WW(N,L)-DW(N,L))/(DW(N,L)-CW(N,L))
      CMPER=CM(N,L)*100.
      WRITE(3,10)L,CMPER
   10 FORMAT(//6X,'MOISTURE CONTENT SECTION :',I3,'=',F8.3,'%')
  300 CONTINUE
      CMAV=((WW(N,1)+WW(N,2)+WW(N,3))-(DW(N,1)+DW(N,2)+DW(N,3)))/((DW(N,
     11)+DW(N,2)+DW(N,3))-(CW(N,1)+CW(N,2)+CW(N,3)))
      WATER=CMAV*100.
      WRITE(3,11) WATER
   11 FORMAT(////36X,'AVERAGE MOISTURE CONTENT =',F8.3,'%')
      WRITE(3,12)N
   12 FORMAT(//6X,'VOID RATIOS OF SAMPLE NO.',I3)
      WRITE(3,14)
   14 FORMAT('0')
      DO 400 L=1,3
      EV(N,L)=SG(N)  *CM(N,L)
      WRITE(3,15)L,EV(N,L)
   15 FORMAT(//6X,'VOID RATIO SECTION:',I3,'=',F8.3)
```

```
  400 CONTINUE
      EVAV=SG(N)*CMAV
      WRITE(3,16)EVAV
   16 FORMAT(////36X,'AVERAGE VOID RATIO =',F8.3)
  100 CONTINUE
      STOP
      END
```

RESULTS OF TRIAXIAL TEST PERFORMED ON 100% SATURATED SAMPLE NUMBER 1

PERCENT OF CARBON SOIL=100.   ANISOTROPIC RATIO= 1.00

VERTICAL CONSOLIDATION PRESSURE= 1.00   LATERAL CONSOLIDATION PRESSURE= 1.

STRAIN PERCEN	AREA SQCM	PORE T KGSQCM	PORE D KGSQCM	SIGMA1 KGSQCM	SIGMA3 KGSQCM	SIG1EF KGSQCM	SIG3EF KGSQCM	DEVSTR KGSQCM	STRESS RATIO	ACOEFF D VALUE
0.0	8.81	3.066	0.0	1.000	1.000	1.000	1.000	0.0	1.000	0.0
0.013	8.81	3.157	0.091	1.092	1.000	1.001	0.909	0.092	1.102	0.985
0.039	8.81	3.209	0.143	1.223	1.000	1.080	0.857	0.223	1.260	0.640
0.065	8.81	3.241	0.175	1.334	1.000	1.159	0.825	0.334	1.405	0.523
0.103	8.82	3.276	0.210	1.443	1.000	1.233	0.790	0.443	1.560	0.474
0.129	8.82	3.297	0.231	1.506	1.000	1.275	0.769	0.506	1.658	0.456
0.168	8.82	3.335	0.269	1.589	1.000	1.320	0.731	0.589	1.805	0.457
0.194	8.82	3.353	0.287	1.633	1.000	1.346	0.713	0.633	1.888	0.453
0.220	8.83	3.375	0.309	1.668	1.000	1.358	0.691	0.668	1.967	0.463
0.259	8.83	3.402	0.336	1.728	1.000	1.392	0.664	0.728	2.096	0.462
0.285	8.83	3.416	0.350	1.760	1.000	1.410	0.650	0.760	2.169	0.461
0.310	8.83	3.433	0.367	1.778	1.000	1.412	0.633	0.778	2.229	0.471

0.737	8.87	3.595	0.529	1.987	1.000	1.458	0.471	0.987	3.097	0.536
1.009	8.90	3.634	0.568	2.041	1.000	1.473	0.432	1.041	3.412	0.546
1.100	8.90	3.644	0.578	2.059	1.000	1.481	0.422	1.059	3.511	0.546
1.229	8.92	3.671	0.605	2.077	1.000	1.472	0.395	1.077	3.724	0.562
1.358	8.93	3.685	0.619	2.091	1.000	1.472	0.381	1.091	3.862	0.567
1.617	8.95	3.725	0.659	2.098	1.000	1.438	0.341	1.098	4.222	0.601
2.135	9.00	3.745	0.679	2.123	1.000	1.444	0.321	1.123	4.498	0.605
2.393	9.02	3.758	0.692	2.123	1.000	1.431	0.308	1.123	4.642	0.616
2.458	9.03	3.759	0.693	2.122	1.000	1.429	0.307	1.122	4.656	0.617
2.717	9.05	3.786	0.720	2.119	1.000	1.400	0.280	1.119	4.992	0.643
3.234	9.10	3.781	0.715	2.116	1.000	1.401	0.285	1.116	4.923	0.641
3.933	9.17	3.801	0.735	2.087	1.000	1.352	0.265	1.087	5.102	0.676
4.787	9.25	3.808	0.742	2.120	1.000	1.378	0.258	1.120	5.340	0.663
5.783	9.35	3.822	0.756	2.114	1.000	1.358	0.244	1.114	5.566	0.679
7.439	9.51	3.843	0.777	2.151	1.000	1.374	0.223	1.151	6.160	0.675
7.762	9.55	3.833	0.767	2.150	1.000	1.382	0.233	1.150	5.938	0.667
9.599	9.74	3.840	0.774	2.153	1.000	1.378	0.226	1.153	6.104	0.672
10.944	9.89	3.850	0.784	2.135	1.000	1.351	0.216	1.135	6.256	0.691
13.157	10.14	3.839	0.773	2.157	1.000	1.384	0.227	1.157	6.093	0.668
14.955	10.36	3.839	0.773	2.198	1.000	1.425	0.227	1.198	6.273	0.645
19.470	10.94	3.839	0.773	2.165	1.000	1.393	0.227	1.165	6.129	0.663
21.022	11.15	3.850	0.784	2.196	1.000	1.412	0.216	1.196	6.536	0.656
22.316	11.34	3.829	0.763	2.193	1.000	1.430	0.237	1.193	6.036	0.639
24.580	11.68	3.833	0.767	2.161	1.000	1.394	0.233	1.161	5.987	0.661

MOISTURE CONTENTS OF SAMPLE NO. 1

MOISTURE CONTENT SECTION : 1= 41.450%

MOISTURE CONTENT SECTION : 2= 41.751%

MOISTURE CONTENT SECTION : 3= 40.941%

AVERAGE MOISTURE CONTENT = 41.395%

VOID RATIOS OF SAMPLE NO. 1

VOID RATIO SECTION: 1= 1.065

VOID RATIO SECTION: 2= 1.073

VOID RATIO SECTION: 3= 1.052

AVERAGE VOID RATIO = 1.064

APPENDIX 3a

SHEAR DATA

Failure Defined at Maximum Effective Stress Ratio
Isotropically Consolidated

$\bar{\sigma}3c$ (kg./sq.cm.)	ε_f (%)	$\bar{\sigma}1-\bar{\sigma}3$ (kg./sq.cm.)	$\bar{\sigma}1$ (kg./sq.cm.)	$\bar{\sigma}3$ (kg./sq.cm.)	$\bar{\sigma}1/\bar{\sigma}3$	$w_{ave.}$ (%)	$e_{ave.}$	u_{wf} (kg./sq.cm.)	$\underline{A}_f$
Untreated Soil					$\Phi' = 32.8$ Degrees				
1.0	21.0	1.20	1.41	0.21	6.54	41.4	1.06	0.78	0.66
2.0	15.1	1.82	2.38	0.56	4.22	36.4	0.94	1.44	0.79
3.0	22.6	2.70	3.62	0.92	3.93	32.8	0.84	2.08	0.77
75% Untreated Soil					$\Phi' = 32.2$ Degrees				
1.0	7.0	0.92	1.21	0.29	4.22	36.7	0.95	0.71	0.78
2.0	13.4	1.85	2.16	0.52	4.53	32.3	0.84	1.48	0.80
3.0	17.1	2.68	2.92	0.89	4.02	31.0	0.80	2.11	0.79
50% Untreated Soil					$\Phi' = 32.2$ Degrees				
1.0	14.8	0.84	1.02	0.18	5.62	37.4	0.98	0.82	0.98
2.0	13.3	1.61	2.16	0.55	3.93	35.5	0.93	1.45	0.90
3.0	21.4	2.31	2.92	0.61	4.81	30.3	0.79	2.39	1.01
25% Untreated Soil					$\Phi' = 25.2$ Degrees				
1.0	18.1	0.83	0.97	0.14	6.60	36.0	0.94	0.85	1.04
2.0	20.3	1.45	2.07	0.62	3.30	33.7	0.89	1.37	0.95
3.0	10.6	2.42	3.62	1.20	3.01	29.8	0.78	1.80	0.75
Treated Soil					$\Phi' = 23.8$ Degrees				
1.0	17.4	0.64	0.95	0.31	3.04	32.4	0.86	0.69	1.07
2.0	23.5	1.17	1.80	0.63	2.88	28.0	0.75	1.38	1.17
3.0	16.4	1.79	2.81	1.02	2.75	26.1	0.69	1.98	1.11

APPENDIX 3b

SHEAR DATA

Failure Defined at Maximum Stress Difference
Isotropically Consolidated

$\bar{\sigma}3c$ (kg./sq.cm.)	ε_f (%)	$\bar{\sigma}1-\bar{\sigma}3$ (kg./sq.cm.)	$\bar{\sigma}1$ (kg./sq.cm.)	$\bar{\sigma}3$ (kg./sq.cm.)	$\bar{\sigma}1/\bar{\sigma}3$	$w_{ave.}$ (%)	$e_{ave.}$	u_{wf} (kg./sq.cm.)	$\underline{A}_f$
Untreated Soil					Φ' = 32.2 Degrees				
1.0	15.0	1.20	1.42	0.22	6.27	41.4	1.06	0.77	0.64
2.0	15.1	1.82	2.38	0.56	4.22	36.4	0.94	1.44	0.79
3.0	22.6	2.70	3.62	0.92	3.93	32.8	0.84	2.08	0.77
75% Untreated Soil					Φ' = 32.3 Degrees				
1.0	12.5	0.95	1.26	0.31	4.01	36.7	0.95	0.69	0.73
2.0	15.1	1.86	2.40	0.54	4.47	32.3	0.84	1.46	0.79
3.0	17.1	2.68	3.57	0.89	4.02	31.0	0.80	2.11	0.79
50% Untreated Soil					Φ' = 32.2 Degrees				
1.0	14.8	0.84	1.02	0.18	5.62	37.4	0.98	0.82	0.98
2.0	13.3	1.61	2.16	0.55	3.93	35.5	0.93	1.45	0.90
3.0	15.2	2.34	2.98	0.64	4.60	30.3	0.79	2.35	1.02
25% Untreated Soil					Φ' = 25.2 Degrees				
1.0	18.1	0.83	0.97	0.14	6.6	36.0	0.94	0.85	1.04
2.0	12.4	1.50	2.16	0.66	3.26	33.7	0.89	1.34	0.89
3.0	10.6	2.42	3.62	1.20	3.01	29.8	0.78	1.80	0.75
Treated Soil					Φ' = 21.4 Degrees				
1.0	2.3	0.68	1.66	0.48	2.42	32.4	0.86	0.52	0.76
2.0	10.3	1.30	2.03	0.73	2.75	28.0	0.75	1.26	0.98
3.0	4.7	1.83	3.11	1.28	2.43	26.1	0.69	1.72	0.94

APPENDIX 3c

SHEAR DATA

Failure Defined at Maximum Effective Stress Ratio
Anisotropically Consolidated

$\bar{\sigma}3c$ (kg./sq.cm.)	ε_f (%)	$\bar{\sigma}1-\bar{\sigma}3$ (kg./sq.cm.)	$\bar{\sigma}1$ (kg./sq.cm.)	$\bar{\sigma}3$ (kg./sq.cm.)	$\bar{\sigma}1/\bar{\sigma}3$	$w_{ave.}$ (%)	$e_{ave.}$	u_{wf} (kg./sq.cm.)	$\underline{A}_f$
Untreated Soil					$\Phi' = 32.2$ Degrees				
1.0	15.6	1.44	1.97	0.53	3.71	37.0	0.95	0.47	0.50
2.0	15.2	2.50	3.52	1.02	3.43	33.5	0.86	0.97	0.65
3.0	20.4	3.69	5.01	1.32	3.80	31.7	0.82	1.68	0.79
75% Untreated Soil					$\Phi' = 32.0$ Degrees				
1.0	16.3	1.23	1.75	0.52	3.38	37.2	0.96	0.48	0.68
2.0	15.7	2.48	3.35	0.87	3.86	32.2	0.83	1.13	0.77
3.0	16.0	3.44	4.81	1.37	3.51	30.0	0.78	1.63	0.84
50% Untreated Soil					$\Phi' = 32.2$ Degrees				
1.0	18.5	1.26	1.74	0.48	3.62	35.1	0.92	0.52	0.68
2.0	18.2	2.05	2.66	0.61	4.34	32.8	0.86	1.39	1.32
3.0	18.5	2.92	4.35	1.43	3.04	30.1	0.79	1.57	1.10
5.0	24.5	4.77	6.59	1.82	3.63	27.1	0.71	3.12	1.40
25% Untreated Soil					$\Phi' = 25.8$ Degrees				
1.0	3.0	1.59	2.13	0.54	3.91	32.9	0.87	0.46	0.42
2.0	20.2	1.92	2.60	0.68	3.81	32.0	0.85	1.32	1.43
3.0	24.4	2.80	4.08	1.28	3.20	30.40	0.80	1.72	1.33
Treated Soil					$\Phi' = 23.8$ Degrees				
1.0	16.2	1.02	1.35	0.33	4.07	31.2	0.83	0.67	1.30
2.0	16.8	1.73	2.50	0.77	3.25	28.4	0.75	1.23	1.68
3.0	19.0	2.87	3.78	0.91	4.14	25.4	0.67	2.09	1.53

APPENDIX 3d

SHEAR DATA

Failure Defined at Maximum Stress Difference
Anisotropically Consolidated

$\bar{\sigma}3c$ (kg./sq.cm.)	ε_f (%)	$\bar{\sigma}1-\bar{\sigma}3$ (kg./sq.cm.)	$\bar{\sigma}1$ (kg./sq.cm.)	$\bar{\sigma}3$ (kg./sq.cm.)	$\bar{\sigma}1/\bar{\sigma}3$	$w_{ave.}$ (%)	$e_{ave.}$	u_{wf} (kg./sq.cm.)	$\underline{A}_f$
Untreated Soil					Φ' = 32.2 Degrees				
1.0	15.6	1.44	1.97	0.53	3.71	37.0	0.95	0.47	0.50
2.0	15.2	2.50	3.52	1.02	3.43	33.5	0.86	0.97	0.65
3.0	23.4	3.69	5.02	1.33	3.77	31.7	0.82	1.67	0.76
75% Untreated Soil					Φ' = 32.2 Degrees				
1.0	20.2	1.23	1.78	0.55	3.26	37.2	0.96	0.46	0.62
2.0	23.1	2.51	3.41	0.90	3.78	32.2	0.83	1.10	0.73
3.0	22.5	3.48	4.89	1.41	3.47	30.0	0.78	1.59	0.80
50% Untreated Soil					Φ' = 32.2 Degrees				
1.0	23.7	1.27	1.77	0.50	3.56	35.1	0.92	0.50	0.65
2.0	13.5	2.08	2.73	0.65	4.21	32.8	0.86	1.35	1.25
3.0	13.3	2.95	4.43	1.48	3.01	30.1	0.79	1.53	1.05
5.0	19.3	4.78	6.63	1.85	4.78	27.1	0.71	3.15	1.38
25% Untreated Soil					Φ' = 25.2 Degrees				
1.0	3.0	1.59	2.13	0.54	3.91	32.9	0.87	0.46	0.42
2.0	0.7	2.23	3.55	1.32	2.70	32.0	0.85	0.68	0.55
3.0	0.7	2.98	5.22	2.24	2.33	30.40	0.80	0.76	0.51
Treated Soil					Φ' = 21.4 Degrees				
1.0	1.8	1.20	1.79	0.59	3.02	31.2	0.83	0.41	0.58
2.0	1.1	1.81	3.24	1.43	2.27	28.4	0.75	0.57	0.71
3.0	0.8	3.23	5.33	2.10	2.54	25.4	0.67	0.90	0.52

BIBLIOGRAPHY

Andersen, A. and Simons, N. E. (1960). Norwegian Triaxial Equipment and Technique. Research Conference on the Shear Strength of Cohesive Soil. A. S. C. E., pp 695-709.

Bishop, A. W. and Henkel, D. J. (1962). The Measurement of Soil Properties in the Triaxial Test. Edward Arnold (Publishers) LTD. Second edition, 1962.

Green, W. J. (1969). The Influence of Several Factors on the Rate of Secondary Compression of Soil. Master's Thesis. University of Missouri at Rolla.

Henkel, D. J. (1960). The Shear Strength of Saturated Remolded Clays. Research Conference on the Shear Strength of Cohesive Soil. A. S. C. E., pp 533-554.

Ladd, C. C. (1964). Stress-Strain Modulus of Clay in Undrained Shear. A. S. C. E. Journal of Soil Mechanics and Foundations Division, September 1964, pp 103-132.

Ladd, C. C. (1965). Stress-Strain Behavior of Anisotropically Consolidated Clays During Undrained Shear. Proc. 6th Int. Conf. Soil Mechanics and Foundations Engineering, Montreal, vol. 1, pp 282-286.

Olson, R. E. (1962). The Shear Strength Properties of Calcium Illite. Geotechnique, vol. 12, no. 1, 1962, pp 23-43.

Rogatkina, Zh. E. (1967). Effect of Anisotropy of Clay Soils on Their Physico Mechanical Properties. Soil Mechanics and Foundations Engineering (translated from Russian). No. 1, 1967, pp 23-26.

Schmidt, N. O. (1965). A Study of the Isolation of Organic Matter as a Variable Affecting Engineering Properties of a Soil. Ph.D. Thesis, University of Illinois, Department of Civil Engineering, Urbana, Illinois.

Schrotberger, D. L. (1966). The Effect of Organic Material on the Shear Strength of A Cohesive Soil. Unpublished report for National Science Foundation. University of Illinois.

Seed, H. B., Mitchell, J. K. and Chan, C. K. (1960). The Strength of Compacted Cohesive Soils. Research Conference on the Shear Strength of Cohesive Soil. A. S. C. E.

Taylor, D. W. (1941). Seventh Progress Report on Shear Research to U. S. Engineers, M.I.T. Publication.

Wascher, H. L., Smith, R. S. and Odell, R. T. (1951). Iroquois County Soils. University of Illinois Agricultural Experiment Station, Soil Report 74.

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